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STATE-OF-THE-ART REVIEW ON D-CRACKING OF CONCRETE IN MANITOBA

PREPARED BY

INRAD INDUSTRIAL RESEARCH AND DEVELOPMENT LTD.

WINNIPEG, MANITOBA

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SUMMARY

STATE OF THE ART REVIEW OF D-CRACKING OF CONCRETE IN MANITOBA

This study outlines existing theories of the nature and causes of D-Cracking and methods of testing concrete and aggregate specimens. It examines the concrete roads and concrete at airports in the Province of Manitoba and makes suggestions for remedial measures to prevent D-Cracking and to prolong the life of pavements. It concludes with recommendations for further research, with the aim to produce concrete that is nonsusceptible to D-Cracking.

RÉSUMÉ

L'ÉTUDE DES CONDITIONS ACTUELLES FENDILLEMENT (D-CRACKING) DU BÉTON ARMÉ AU MANITOBA

Cette étude décrit les theories courant de la nature et les causes de fendillement (D-Cracking) du béton armé et les méthodes d'essai du béton et spécimens d'agrégat. Les données décrit les routes bétoné et les bétons au aeroports au Manitoba et recommandations sont faite pour préserver la longévité des pavements. En conclusion, des recherche est proposé de faire du béton non susceptible de fendillement (D-Cracking).

CHAPTER 1

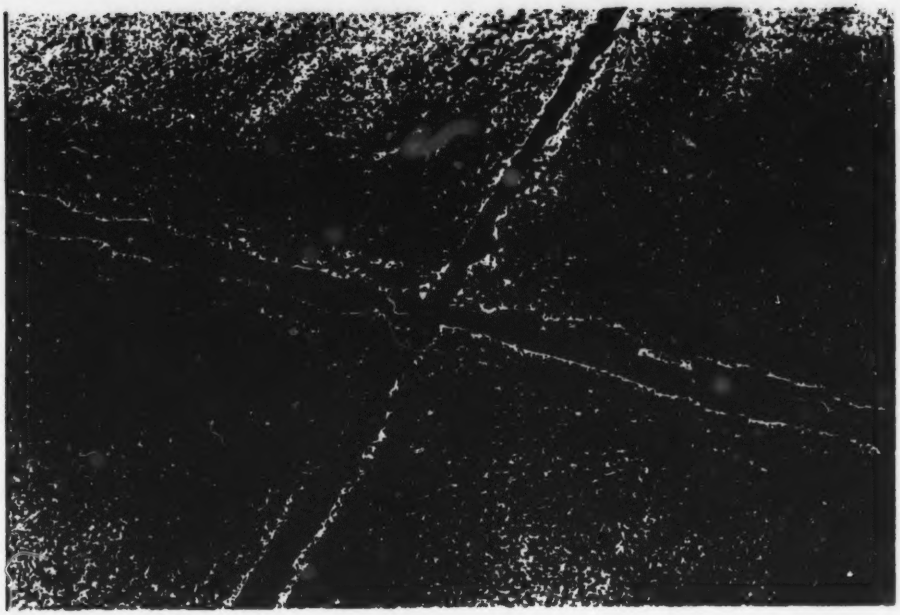
D-CRACKING OF CONCRETE

1.0 INTRODUCTION

This section is devoted to a discussion of the nature of D-Cracking, the causes, and the role that aggregates play in the deterioration process. The discussion is based on information published in technical articles. References from which the information is drawn are cited in the text. Additional sources of relevant information appear in the Bibliography.

1.1 Nature of D-Cracking

The phenomena of D-Cracking, a type of concrete deterioration, has been defined as "fine, closely spaced cracks which occur parallel and adjacent to longitudinal and transverse joints, intermediate cracks and the free edges of pavement slabs" (Stark, 1976). Two examples of well developed D-Cracking patterns are shown in Figure 1.1. Figure 1.1a is a photograph of the pattern developed at the intersection of four interior slabs, on a runway at the airfield at Rivers, Manitoba. The airfield had been abandoned for many years, permitting the D-Cracking process to continue without the weakened concrete being broken up by traffic. Thus the pattern of crack development is well defined and the picture portrays a classical example of D-Cracking. Figure 1.1b is a photograph of D-Cracking that has developed at the intersection of a structural crack and a transverse joint, on a street in Winnipeg. The pattern is basically very similar to that observed at interior joints.



(a)

(b)

Figure 1.1 Examples of D-Cracking patterns: (a) intersection of construction joints, (b) intersection of a structural crack and a transverse joint.

According to Girard et al, 1982, the "finely spaced cracks can continue around corners and progress toward the centre of the slab". The progression of the fine cracks away from the joints is illustrated in Figure 1.2 (Rivers airfield), and into the interior of the slab in Figure 1.3 (Churchill airfield).

The phenomenon does not necessarily include all the slabs at a joint intersection as illustrated in Figure 1.4 (Churchill). In this case the D-Cracking involved the two slabs of the common lane and did not involve the two slabs of the adjoining lane. In Figure 1.5 (Rivers), it can be seen that the deterioration is severe in only one of the four slabs, although the other three slabs do show evidence of minor deterioration.

Where the concrete undergoing D-Cracking is subjected to traffic, the pattern is usually less recognizable. Figure 1.6a shows an example of D-Cracking on a Winnipeg residential street where the traffic is light. The pattern is recognizable, but not well developed. Figure 1.6b shows an example of D-Cracking on Hwy No. 2, which is a heavily travelled highway. Here there are no fine cracks, but the pattern of break-up of the concrete does resemble the pattern of fine cracks associated with classical D-Cracking.

During the survey of the streets of Winnipeg, it was found that D-Cracking was also evidence around manholes which were commonly located along the street centreline. Examples are shown in Figure 1.7. The D-Cracking was generally more severe at the manholes than at the construction joints.

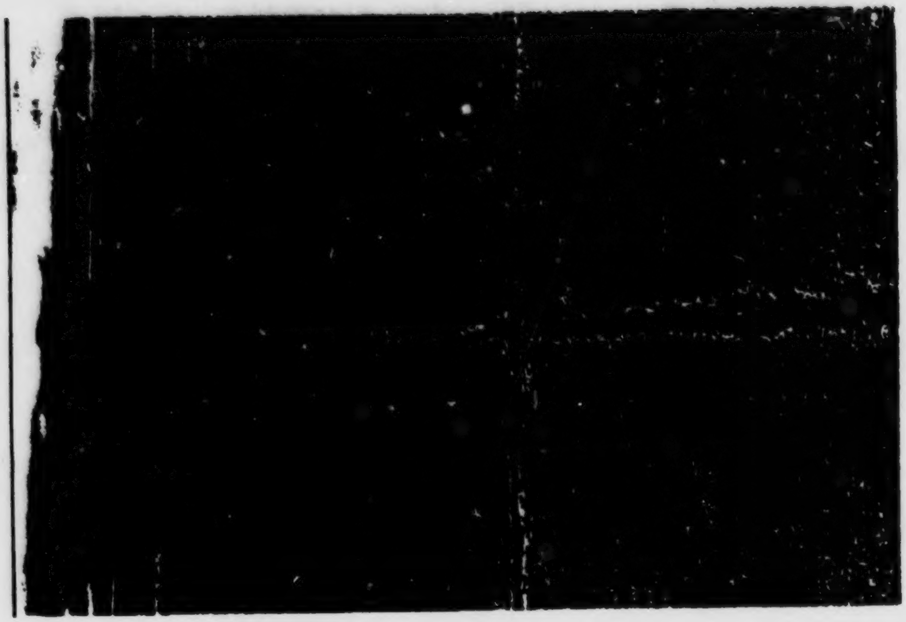


Figure 1.2 Progression of cracks away from joint intersection

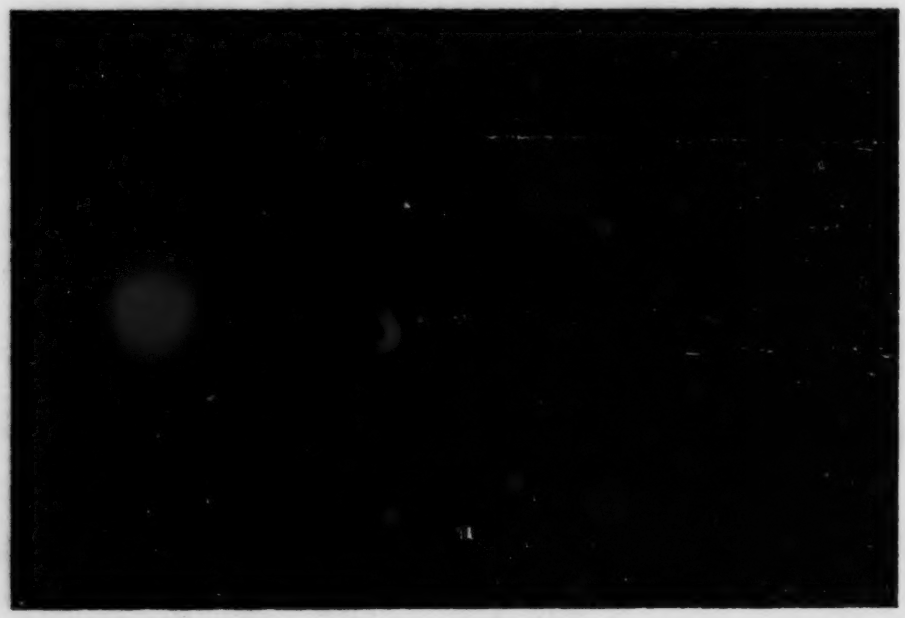


Figure 1.3 Progression of cracks into slab interior

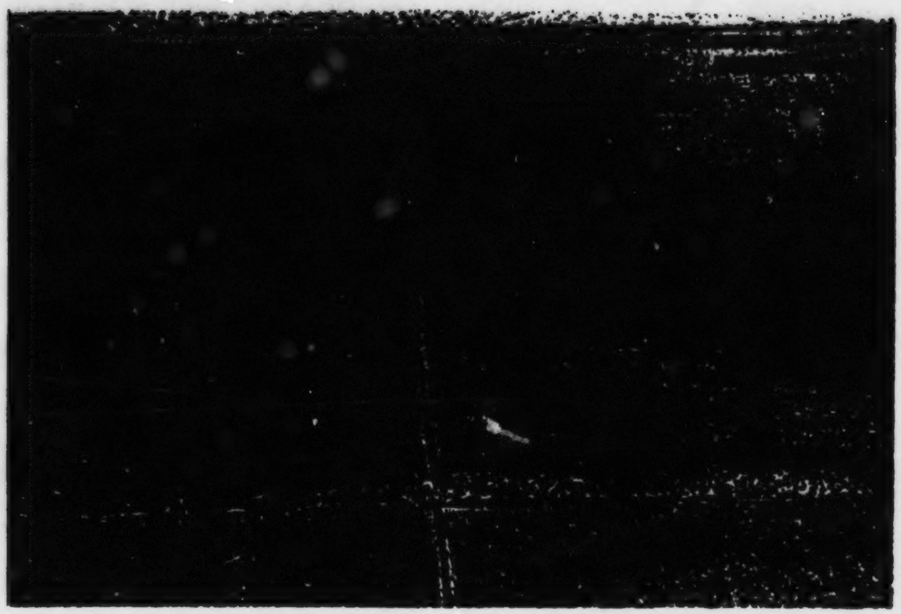


Figure 1.4 Deterioration limited to one lane

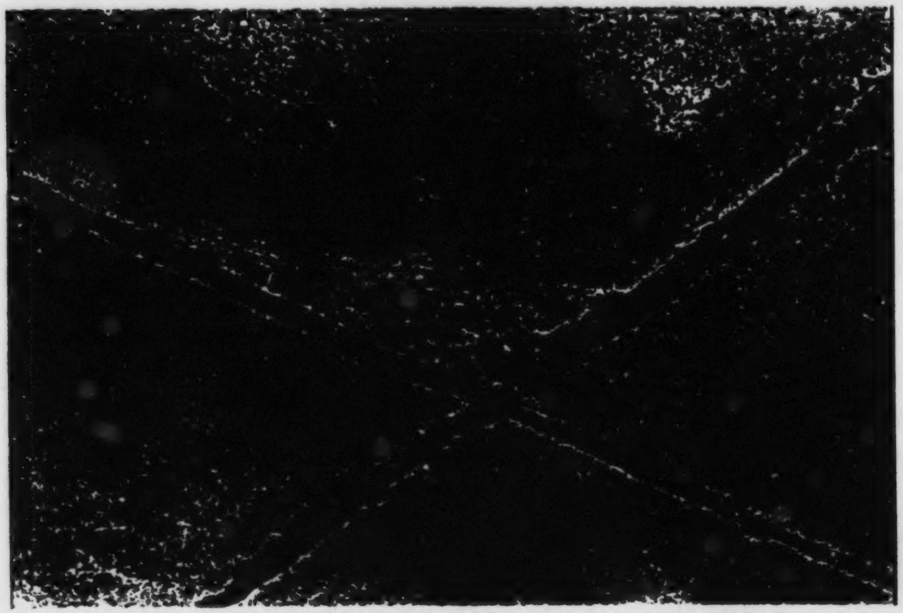
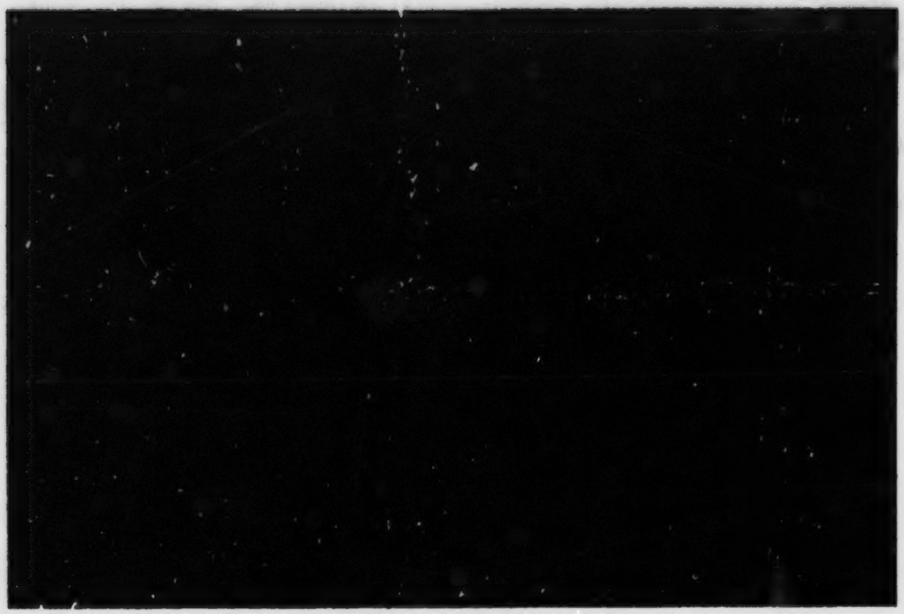
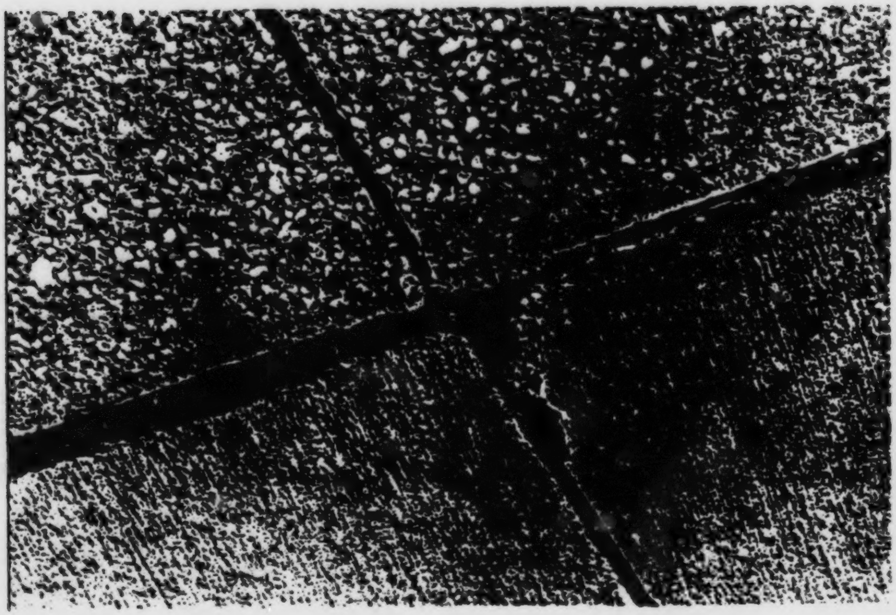


Figure 1.5 Severe deterioration limited to one slab



(a)



(b)

Figure 1.6 Examples of poorly developed D-Cracking patterns: (a) residential street, (b) provicial highway

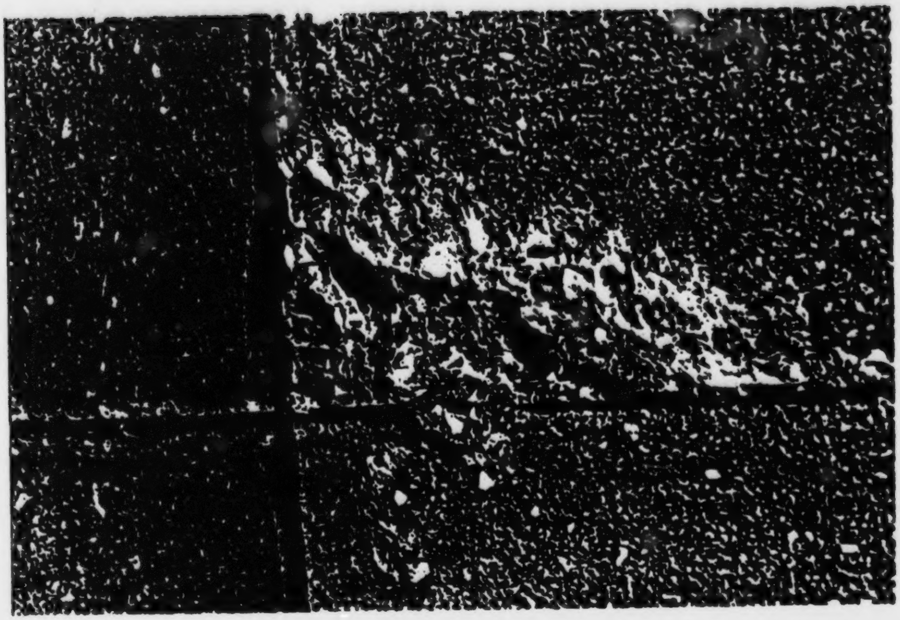
Figure 1.7 Examples of concrete deterioration around manholes

1.2 Causes of D-Cracking

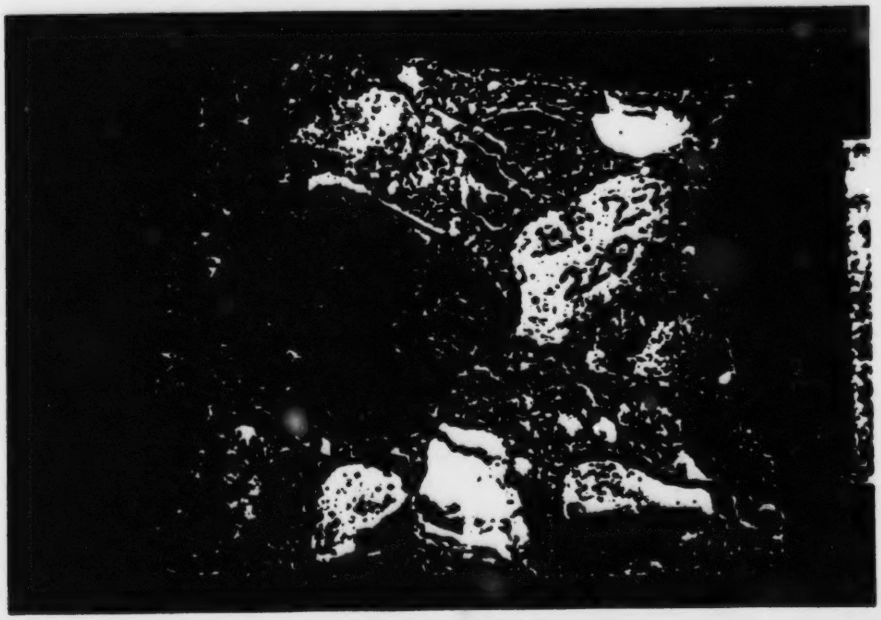
Literature pertaining to the causes of D-Cracking originated primarily from studies of the problem in the mid-western United States. Field studies were undertaken to determine the factors associated with the problem. Girard et al (1982) identified the following two factors, as those that played a dominant role in D-Cracking:

1. The coarse aggregate fraction of the concrete
2. A freeze-thaw environment

The importance of the aggregate in the frost resistance of concrete has been documented over the years by investigators such as Powers (1955), Verbeck & Landgren (1960), Stark & Klieger (1973), Bukovatz et al (1974), Stark (1976), Winslow et al (1982), and Traylor (1982). The studies showed that most aggregates associated with D-Cracking were sedimentary fine-grained carbonates or silicious rock (Stark, 1976). It is generally accepted that the deterioration of the concrete is initiated by fracture of the coarse aggregate particles. Examples of D-Cracking, where several fractured aggregates have been exposed, are shown in Figure 1.8. Figure 1.8a is a pavement joint on a street in Winnipeg, and Figure 1.8b is a specimen used in a laboratory study of D-Cracking at the University of Manitoba. According to Powers (1955), the important aggregate properties insofar as D-Cracking is concerned, are porosity, permeability, and pore-size distribution. A more detailed discussion of the influence of the aggregate pore structure will be presented in a later section.



(a)



(b)

Figure 1.8 Examples of fractured aggregates: (a) city street, (b) laboratory specimen

It has been established that a freeze-thaw environment is essential to the D-Cracking process. Laboratory studies have demonstrated that deterioration of concrete specimens is directly related to the number of freeze-thaw cycles. The effects of the rate of freezing on the frost resistance of concrete has not been conclusively ascertained. The work of Powers (1945) and Litvan (1972) suggests that a fast cooling rate is more detrimental than a slow cooling rate, whereas the work of Collins (1944) and Lin & Walker (1975) suggests the opposite. In any event, D-Cracking has occurred in climates ranging from relatively "mild", as for example in Missouri, U.S.A., to "extremely cold", as for example in Churchill, Manitoba. Although Janssen & Dempsey (1985) used four different cooling rates in their D-Cracking study, there were other variables involved which did not permit them to determine the influence of the cooling rate on the severity of deterioration.

It is also widely accepted that moisture is a very important factor in the D-Cracking process. Stark (1976) observed that D-Cracking is usually initiated in the lower third of a pavement slab where moisture is readily available from the subbase. The observation that aggregates known to be susceptible to D-Cracking in concrete pavements, performed satisfactorily in bridge decks (Axon et al, 1969), lends credence to the suggestion that external moisture is essential to the process. Stark (1970) found field evidence that poorly drained subbases were associated with D-Cracking and that the installation of longitudinal drains delayed and, in some instances eliminated, the problem. However, Traylor (1982) stated that drainage and the type of subbase did not significantly affect the severity of D-Cracking. Both Stark (197) and Girard et al (1982) found that placing a polyethylene vapor barrier between the concrete and the subbase promoted D-Cracking.

Thus, studies have established that a freeze-thaw environment and coarse aggregate susceptible to fracturing under repeated freeze-thaw cycles, are essential to the D-Cracking process. The question of whether or not access to external moisture is an essential requirement, or simply an influential factor, has not been ascertained unequivocally. Other factors that could affect the occurrence and severity of D-Cracking noted by Girard et al (1982) were:

1. Type of subgrade soil
2. Properties of the fine aggregate
3. Properties of the cement
4. Proportioning of the components of the concrete
5. Quality of the concrete
6. Method of curing
7. Design of the pavement structure
8. Weather conditions during construction
9. Traffic and loading conditions

1.3 Frost Action in Aggregates

A considerable amount of research has been done on durability of concrete and concrete aggregates in a freeze-thaw environment. According to Verbeck and Landgren (1960), the freeze-thaw durability of concrete aggregates depends on the rate at which the aggregates become critically saturated, which in turn depends on the pore properties of the paste as well as the aggregate, and the vapor pressure within the aggregate pores. In this respect fine-grained aggregates achieve a higher degree of saturation than do coarse-grained aggregates which would tend to make them more susceptible to D-Cracking. The temperature at which water freezes in aggregates also depends on pore size since small pores cause a depression of the freezing point. Water in coarse-grained rock does not experience a significant freezing point depression, whereas in fine-grained rock, nucleation may not begin until the temperature drops to -5° to -7°C (Verbeck & Landgren, 1960; Dunn & Hudec, 1972).

Verbeck and Landgren (1960) stated that a saturated aggregate may cause concrete to fracture upon freezing if the aggregate does not meet the following three requirements:

1. It must be able to elastically accommodate the volume increase associated with phase change.

2. It must not fail during the freezing process due to critical size effects. (That is, the combination of porosity and permeability must be such that very high hydraulic pressures are not developed, for example, in the case of high porosity and low permeability).

3. It must not have a combination of porosity and permeability such that very high hydraulic pressures are developed at the aggregate paste interface, that is, high porosity and high permeability in the aggregate and low permeability in the paste.

Marks and Dubberke (1982) found, by using scanning electron miscroscope techniques, that durable aggregates are either coarse-grained or extremely fine-grained.

The exact mechanism by which aggregates are fractured in a freeze-thaw environment has not been established. Four possible mechanisms have been postulated. These are summarized below.

1.3.1 Hydraulic pressure theory

The "hydraulic pressure theory" proposed by Powers (1955) states that, if the degree of saturation of an open pore is greater than 91.7%, water will be forced to move out of the pore during the freezing process. This is caused by the 9% expansion of water, associated with phase change. Resistance to this flow generates hydraulic pressure in the pore, which may exceed the tensile strength of the material.

The magnitude of hydraulic pressure generated in an aggregate will depend on the degree of saturation, viscosity of the water, rate of ice formation, the permeability of the aggregate, and the distance to the nearest escape boundary. The distance, of course, depends on aggregate size and hence the role of aggregate size in the D-Cracking process is obvious in the context of the hydraulic pressure theory. Verbeck and Landgren (1960) calculated the critical size for a chert and for a dolomite, under certain conditions of freezing and no confinement, to be 13 mm and 840 mm respectively, illustrating the relative susceptibility of the two.

1.3.2 Free Energy Difference theory

The Free Energy Difference theory was proposed by Powers and Helmuth (1953) after it was observed that the hydraulic pressure theory could not account for all the phenomena observed when cement paste was subjected to freezing and thawing. It was found that dilation could occur even when no additional ice was being formed and that freezing of capillary water did not result in expansion. These phenomena were attributed to the difference in free energy between water and ice. When ice forms in a capillary, the thermodynamic equilibrium of the system is disturbed because the entropy of ice is less than that of adsorbed water. As the temperature decreases, the difference in free energy between the two phases increases, creating an energy gradient which causes the water in the cement gel to flow either into gel pores, if they exist, or into the capillary cavities of the cement paste, where it freezes, enlarging the cavity. Also, the free energy of the capillary ice is higher than that of the air void ice because of capillary and pressure effects. Moreover, the free energy of the capillary ice continues to increase because of the pressures applied by the pore walls as its volume increases due to the influx of gel water. Consequently, at some point the free energy of the capillary ice becomes equal to that of the gel water and the direction of flow is reversed. The magnitude of hydraulic pressure thus depends on the amount of water that flows into the capillaries which, in turn, depends on the availability of gel pores for accommodating the flow induced by energy gradients. The addition of air entrainment provides the cement paste with gel pores, thereby reducing or even eliminating the build up of hydraulic pressures.

The above phenomenon requires freezing point depression in the capillaries. In coarse-grained aggregates there is virtually no depression of the freezing point and consequently induced flows are essentially negligible. However, in fine-grained aggregates, the pores may be small enough to cause a significant freezing point depression and there is unfrozen water in contact with ice, within the pores. This results in an imbalance of free energies within the aggregate which increases the hydraulic pressure. Thus, the pressures acting on the walls of the aggregate pores consist of hydraulic pressures plus the pressure exerted by the growth of ice crystals within the pores.

1.3.3 Desorption theory

A desorption or vapor pressure theory was proposed by Litvan (1972, 1978), postulating that the mass movement of water is due to the existence of nonequilibrium thermodynamic conditions between the capillary ice and the water in the gel pores, rather than due to volume expansion associated with phase change.

In order that a porous body remains saturated, the humidity must be 100%. Since there is a significant depression of the freezing temperature in capillary pores, the vapor pressure of the supercooled water and the bulk water, with which it is in contact, must be the same. However, some of the vapor will condense on the pore walls and form ice. The vapor pressure must therefore be lowered to that of ice and the degree of saturation must also decrease. The degree of saturation is lowered by virtue of water being expelled from the capillary pores, the difference in vapor pressure between ice and the supercooled water being the driving force. The expulsion of water creates a hydraulic pressure. The magnitude of the pressure will depend on whether the water must move to a free boundary, or if it can move into an air bubble provided through air entrainment. Thus, the conditions prior to freezing dictate the amount of water that must be expelled, rather than volume changes associated with phase change. Litvan (1978) found that, in slowly cooled samples, the amount of expelled water was 3 to 4 times that associated with expansion due to phase change.

Freezing point depression is a requirement in this postulation and therefore the phenomenon is limited to fine-grained aggregates.

1.3.4 Dual Mechanism theory

The Dual Mechanism theory was proposed by Larson & Cady (1969), and it combines the Powers hydraulic theory with an ordering process proposed by Dunn & Hudec (1972). In the theory, the unfrozen water becomes ordered by adsorption forces onto the surfaces of the mineral and the ice. The ordering process involves volume increases responsible for expansive forces after the ice has been formed. Thus, the expansive forces are the direct result of adsorption rather than desorption.

1.4 Experimental Frost Shattering of Rock

1.4.1 Introduction

Since D-Cracking is attributed to fracture of the coarse aggregate, studies of rock fracture caused by freeze-thaw cycles are relevant to the D-Cracking problem. A study of the breakdown of rock due to freeze-thaw cycles has been underway at the Centre de Geomorphologie du Centre National de la Recherche Scientifique (CNRS), France, since 1965. Their findings, after 15 years of research, were reported by Lautridou & Ozouf (1982). The following is a brief summary of that study.

1.4.2 Test Procedure

As many as one thousand rock samples were subjected to freezing and thawing cycles each year. The rock samples were 90 mm cubes. After saturation under pressure, each cube was placed in a small plastic basin, filled with water to a depth of 10-20 mm, then frozen in a period of about 8 hours. The number of freeze-thaw cycles varied from 500 to 1320.

1.4.3 Results

Some of the results pertinent to the D-Cracking problem were as follows:

1. Rate of freezing played a far more important role in frost shattering than did the intensity of freezing. The rate of freezing controls the movement of an ice-water mixture from the interior of a specimen towards its surface during freezing. This process is known as "cryosuction".

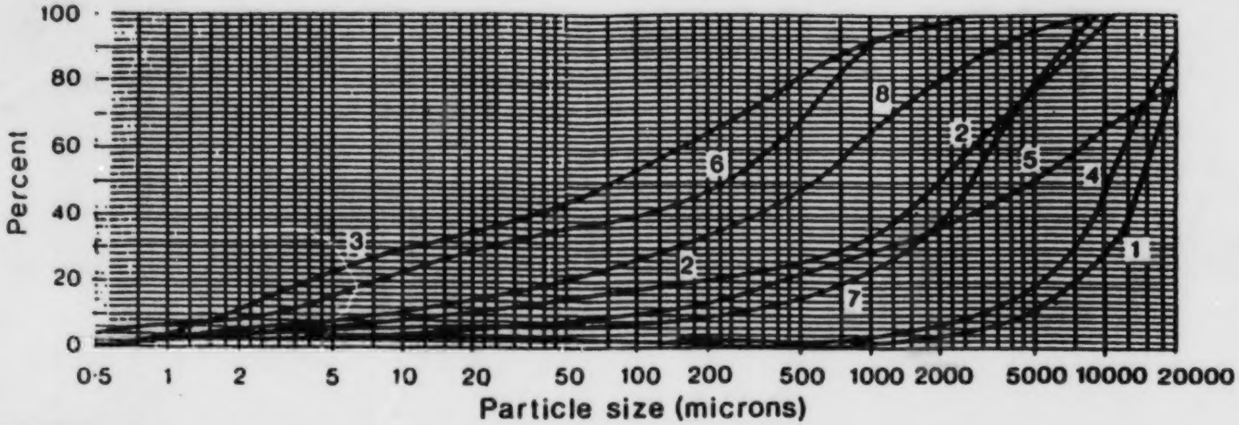


FIGURE 1.9

Grain size distribution curve of rock debris produced by frost shattering (with freezing to a temperature of −8°C) 1: lithographic limestone from Sonneville (Charentes), after 500 freeze-thaw cycles; 2: chalky limestone from Caen (Lower Normandy), after 500 cycles; 3: chalk from Tancarville (Normandy), after 500 cycles; 4: Precambrian schist/siltstone from Lengronne (Lower Normandy), after 1000 cycles; 5: Corsican schist, after 920 cycles; 6: argillite from Hyenville (Lower Normandy), after 1320 cycles; 7: weathered schist from Mesnil-Herman (Normandy), after 1320 cycles; 8: weathered granite from Grand-Celland (Lower Normandy), after 1320 cycles (after Lautridou & Ozouf, 1982).

2. The degree of fragmentation was influenced primarily by the number of freeze-thaw cycles rather than the rate or intensity of freezing.

3. Rocks with a porosity of less than 6% were not particularly prone to frost shattering, but could produce small amounts of weathered debris by superficial damage if the porosity was between 3 and 6%.

4. Rocks which were very porous and very permeable were not prone to frost shattering. (Water is readily displaced from the pores during the freezing process).

5. The shape and slope of the capillary rise curve can be used to characterize the varying degrees of frost susceptibility. This technique offers an inexpensive alternative to such tests as the mercury porosimetry test.

6. Current postulated mechanisms of frost shattering which are based on the notion of hydraulic pressures and water absorption could not fully explain the frost shattering phenomenon.

7. Frost shattering was not only a process of fracturing, but also scaling in the case of limestones and granular disintegration in the case of sandstones and weathered granites.

The breakdown of various rocks as the result of frost shattering is shown in Figure 1.9, which was reproduced from Lautridou & Ozouf (1982). This graph may be useful in assessing the fracture potential of certain aggregates.

1.5 Freeze-Thaw Testing of Aggregates

1.5.1 Sulphate Soundness Test

The purpose of the sulphate soundness test, ASTM C88, is to determine the potential resistance of the aggregate to weathering, which includes a freeze-thaw environment. The aggregate is immersed in a solution of sodium (or magnesium) sulphate for 16 to 18 hours, then dried in an oven. This is repeated 5 to 10 times, and the weight loss is determined. A maximum permissible weight loss is specified. Correlations between this test and field performance of aggregates has been very poor (Roger et al, 1988).

1.5.2 Iowa Freeze-Thaw Test

The Iowa freeze-thaw test (Iowa, 1977) is commonly used as an alternative to the sulphate soundness test. In this test the aggregate is vacuum saturated and subjected to either 16 freeze-thaw cycles in a 0.5% water-alcohol solution or, alternatively, 25 cycles in plain water.

1.5.3 Ontario Ministry of Transportation Test

This test, which is still in the development stage, is essentially a modified version of the Iowa freeze-thaw test. Its development is described by Roger et al (1988). It differs from the Iowa test in that vacuum saturation is not used, the aggregate is soaked in a sodium chloride solution, and the freeze-thaw conditions are different. The effects of solution concentration, the number of freeze-thaw cycles, freezing temperature, temperature variation, and rate of cooling were recently studied, and the following test procedure has been adopted:

The aggregate is placed in autoclave plastic Mason jars and soaked in a 3% by mass sodium chloride solution for 24 hours. The aggregate is then drained and subjected to 5 freeze-thaw cycles. The freezing is done in a -18±2° freezer for 16±2 hours, and the thawing is at room temperature for about 8 hours. After 5 cycles the aggregate is oven dried, sieved, and the loss is expressed as percent by mass of the original sample mass.

1.5.4 University of Manitoba study

In the University of Manitoba study, effects of both vacuum and pressure saturation on the frost resistance of coarse aggregate were studied. The details are included in the next chapter which deals exclusively with all the studies on D-Cracking that were carried out in Manitoba.

1.6 Detecting D-Cracking by Ground Penetrating Radar Equipment

Ground penetrating radar equipment consists of a transmitter, an isolator, antenna, a receiver, a signal processor, and a display unit. Basically, the transmitter generates a "short pulse radar waveform; the isolator provides a direct path from the transmitter to the antenna; the antenna provides a smooth electromagnetic transition from the transmitter to the ground, and it directs the radiated electromagnetic into the ground in a desirable pattern. The wave travels in the radiated direction until it strikes a discontinuity in the electromagnetic properties of the media. A portion of the wave travels through the discontinuity, and a portion is reflected in other directions. The portion of the wave reflected by the discontinuity forms the basis for target detection and assessment.

The ground penetrating radar equipment may provide a means of detecting areas of concrete "deterioration" under asphaltic concrete overlays. Apparently, some studies have been conducted in the United States. Transport Canada is planning to carry out a very limited experimental study of this technique in Winnipeg.

CHAPTER 2

MANITOBA STUDIES

2.0 INTRODUCTION

This chapter describes the studies pertaining to D-Cracking that were carried out at the University of Manitoba. The studies consist of the following:

1. Field surveys to determine the extent and the severity of the problem in Manitoba.

2. A correlation study of pavement performance and aggregate source.

3. Identifying aggregates susceptible to D-Cracking by means of ASTM C-666 B test.

4. Determining the effect of down-sizing aggregates known to be susceptible to D-Cracking.

5. Studying the effect of air entrainment on D-Cracking.

6. Determining whether or not external moisture is essential to the D-Cracking process.

7. Pore structure analyses of durable and nondurable aggregates by various techniques.

8. Petrographic analysis of some Manitoba aggregates known to be susceptible to D-Cracking.

9. Studying the resistance of aggregates to freeze-thaw cycles.

The results of the first two studies (field survey and correlation of field performance and aggregate source) carried out by the University of Manitoba are combined with similar studies conducted as part of the current contract, and are presented in a subsequent chapter. The rest of the studies are presented in separate sections of this chapter. Most of these studies were carried out at the University of Manitoba under the supervision of Dr. Len Domaschuk and his coworkers. Therefore, most of the text that follows (which was prepared by Dr. Domaschuk) was taken from those publications. Some portions were paraphrased, while others were reproduced verbatim. The source for each study is identified, in brackets, immediately after the title of the study and that particular reference is not cited again in the text.

2.1 Identifying Aggregates Susceptible to D-Cracking and the Effect of Maximum Aggregate Size (Domaschuk, Hermanson and Ganapathy, 1986).

These two studies were carried out simultaneously (since they involved the same aggregate sources), and therefore the results of the two studies are presented together in this section.

2.1.1 Aggregates tested

Aggregates for the studies were provided by two local major concrete suppliers. The aggregate sources were 3 pits commonly used for concrete production, in and around Winnipeg. The pits were: Birds Hill (CN), Gull Lake, and Ritcher. The mineralogy based on pebble counts, top size, and absorption of the aggregates are given in Table 2.1.

The aggregate particles from the Birds Hill and Ritcher pits were predominantly carbonate (60 to 75%), and 23 to 35% were of igneous origin. The aggregate particles from the Gull Lake pit were predominantly igneous (67%), with only 28% being carbonates.

Aggregate top-size ranged from 40 mm to 14 mm. Two significantly different top-sizes were selected for both the Birds Hill and the Ritcher pits to examine the effect of top-size on durability.

Table 2.1: Aggregate properties (after Domaschuk et al., 1986)

PIT	DESIG.	MINERALOGY Carb.	Ign.	Other	TOP-SIZE (mm)	ABSORPTION (%)
Birds Hill CN	BP1	75	23	2	20	2.02
Birds Hill CN	BP2	62	31	7	40	1.50
Gull Lake	BP3	28	67	5	20	1.70
Ritcher	S1	70	27	3	28	1.58
Ritcher	S2	59	35	6	14	2.40

The absorption of the aggregates ranged from 1.50% to 2.40%. There was no apparent difference in the absorption of the predominantly carbonate aggregates as compared to the predominantly igneous aggregate.

Fine aggregate from the Birds Hill CN pit was used for all the concrete mixes.

2.1.2 Concrete Mix

The concrete mix design was that used for concrete pavements by Transport Canada (Winnipeg), sponsors of this particular study. The specifications were:

water/cement ratio = 0.45
air entrainment = 5% ± 1%
slump = 50 mm ± 10 mm
28-day strength = 25 MPa

The "as batched" concrete properties are given in Table 2.2.

Table 2.2: Properties of concrete mix (After Domaschuk et al., 1986)

AGGREGATE SOURCE		TOP-SIZE (mm)	SLUMP (mm)	AIR (%)	28-DAY STRENGTH (MPa)
Birds Hill	BP1	20	80	5.9	34.8
Birds Hill	BP2	40	50	4.7	27.7
Gull Lake	BP3	20	80	6.3	30.1
Ritcher	S1	28	40	5.9	26.2
Ritcher	S2	14	60	4.0	43.8

Although the same mixture design was used for all batches, there was considerable variation in the slumps and the strengths between batches. In the case of both the Birds Hill and the Ritcher aggregates, the smaller top-size resulted in significantly higher slumps and strengths, as compared to the larger top-size.

2.1.3 Test specimens and procedure

Three concrete prisms, measuring 76 x 76 x 350 mm, were cast from each batch and allowed to cure for 28 days.

The test was run in accordance with the ASTM C-666 B test procedure with a slight modification. Instead of monitoring the change in dynamic modulus of the specimens, as required by the standard procedure, the change in specimen length was measured. This modification was suggested by Verbeck et al. (1972) and Paxton (1982). They found that the dynamic modulus was not sensitive enough to freeze-thaw deterioration of the coarse aggregate, to serve as a diagnostic parameter of D-Cracking susceptibility. The procedure consisted of:

(i) freezing the specimens in air (-18°C) and thawing them in water (4°C);

(ii) subjecting the specimens to 300 cycles (50 days);

(iii) taking specimen lengths every 30 cycles.

Specific details of the test may be found in the thesis authored by Hermanson (1987).

2.1.4 Test results

The changes in prism lengths were converted to linear strains on the basis of the original lengths of the prisms. A plot of linear strains (average of the 3 prisms) versus the number of freeze-thaw cycles is shown in Figure 2.1 for each aggregate source.

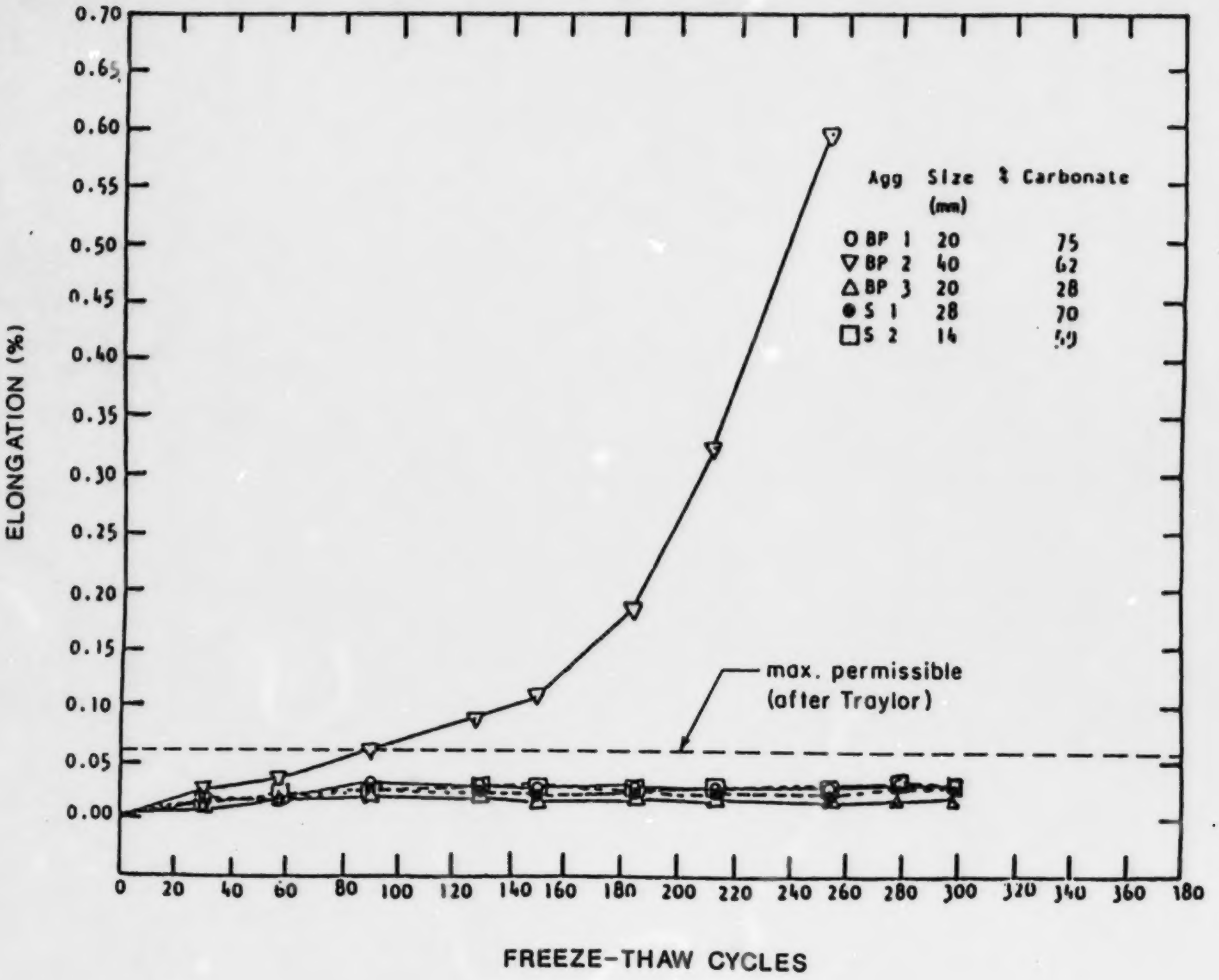


Figure 2.1 Freeze-thaw test data (after Domaschuk et al., 1986)

According to previous correlation studies between prism elongations and D-Cracking susceptibility (Traylor, 1982), elongations in excess of 0.20% were always associated with poor field performance. The lower boundary of elongations associated with D-Cracking varied from 0.06% to 0.10% in his tests. According to Paxton (1982), D-Cracking was not associated with concrete which underwent elongations of less than 0.02% at 300 cycles.

As seen in Figure 2.1, only the prisms made from Birds Hill CN aggregate with a top size of 40 mm exhibited excessive expansion. In fact, the 3 specimens made with this aggregate disintegrated at approximately 250 cycles. Inspection of the disintegrated prisms revealed fractured coarse aggregate particles, which is a characteristic feature of D-Cracking. Thus, of the 5 aggregates investigated, only Birds Hill aggregate with a top-size of 40 mm, was implicated as being D-Cracking susceptible according to the ASTM test, and the Traylor "lower" allowable expansion limit of 0.06%. The linear expansion of the rest of the prisms ranged between 0.02% (Gull Lake, 14 mm) and 0.05% (all other prisms). According to the Paxton criterion, with the exception of the Gull Lake aggregate, the possibility of the other aggregates being D-Cracking susceptible cannot be ruled out entirely. However, the degree of susceptibility of these aggregates was certainly lower than that of the Birds Hill aggregate with a top-size of 40 mm.

The influence of aggregate size on D-Cracking susceptibility could only be ascertained by comparing the performance of the prisms, made with Birds Hill CN aggregate, having top sizes of 40 and 20 mm. Whereas, a top-size of 40 mm resulted in excessive expansion, a reduction in top-size to 20 mm reduced the expansion to 0.05% which can be categorized as marginally susceptible. The reduction in top-size of the Ritcher aggregate, from 28 mm to 14 mm, did not have a discernible effect on the expansion of the prisms.

From the point of view of mineralogy, the predominantly igneous aggregate (Gull Lake) performed only marginally better than did the predominantly carbonate aggregate of comparable top-size (Birds Hill CN, 20 mm).

2.1.5 Conclusions

1. Aggregates highly susceptible to D-Cracking can be positively identified by the ASTM C-666 B test. The test may not be adequate for aggregates with a low degree of susceptibility.

2. Reducing the top-size of the Birds Hill aggregate from 40 mm to 20 mm reduced the linear expansion of the prisms from a magnitude in excess of 0.6% to a magnitude of 0.05%. This changed the susceptibility from "very high" to "marginal".

A more detailed description of this particular study is provided by Hermanson (1987).

2.2 Effect of Air Content (Domaschuk, Ganapathy and Triffo, 1988)

2.2.1 Aggregates

The coarse aggregate used in this study was the Birds Hill BP aggregate with a top size of 40 mm. As mentioned previously, it was susceptible to D-Cracking. The fine aggregate was from the same source as used in the previous studies.

2.2.2 Concrete Mixtures

Since the variable of interest in this study was air content, it would have been desirable to keep all other constituents constant. However, because air content affects the properties of the concrete in both the fresh and hardened states, minor changes in cement and aggregate contents were made in an attempt to meet the Transport Canada specifications. The water-cement ratio (0.45) was kept constant. Three batches of concrete, in which the main variable was air content, were prepared. In one batch no air entrainment agent was added and the air content, i.e. the entrapped air, was 1.9%. To the other two batches, air entrainment was added so as to produce mixtures with 5.0% and 8.5% air contents. The three mixtures are identified in the text by the addition of the air content, as a suffix to the latter "A".

The properties of the fresh and hardened concrete mixtures are listed in Table 2.5. As seen in Table 2.5, increasing the air content had the effect of significantly increasing the slump and decreasing the strength of the concrete. The concrete with 8.5% air did not quite meet the slump and strength requirements specified by Transport Canada.

Table 2.5: Properties of concrete used in the air content study

PROPERTIES	A1.9	A5.0	A8.5
Water	162.4	158.2	141.3
Cement	361.0	351.5	314.1
Crse Agg (SSD)	1140.0	1147.5	1057.3
(dry)	1121.0	1129.0	1057.8
Fine Agg	768.0	697.5	740.2
Air added (ml/cu m)	0.0	200.0	445.0
Air content (%)	1.9	5.0	8.5
Unit wt	2441	2389	2279
Slump (mm)	45	50	80
28-day strength (MPa)	42.0	38.5	22.5

Note: All weights in kg/cu m

2.2.3 Freeze-thaw test procedure and results

For the freeze-thaw tests, 3 prisms were cast from batches having air contents of 1.9% and 8.5%, and 2 prisms were cast from the batch having 5.0% air. The results of the freeze-thaw tests are shown in Figure 2.4, which is a plot of the average elongation of the prisms, as a function of the number of freeze-thaw cycles. It is seen that the linear expansion of the concrete increased substantially as the air content decreased. Elongations of the prisms having air contents of 1.9% and 5.0%

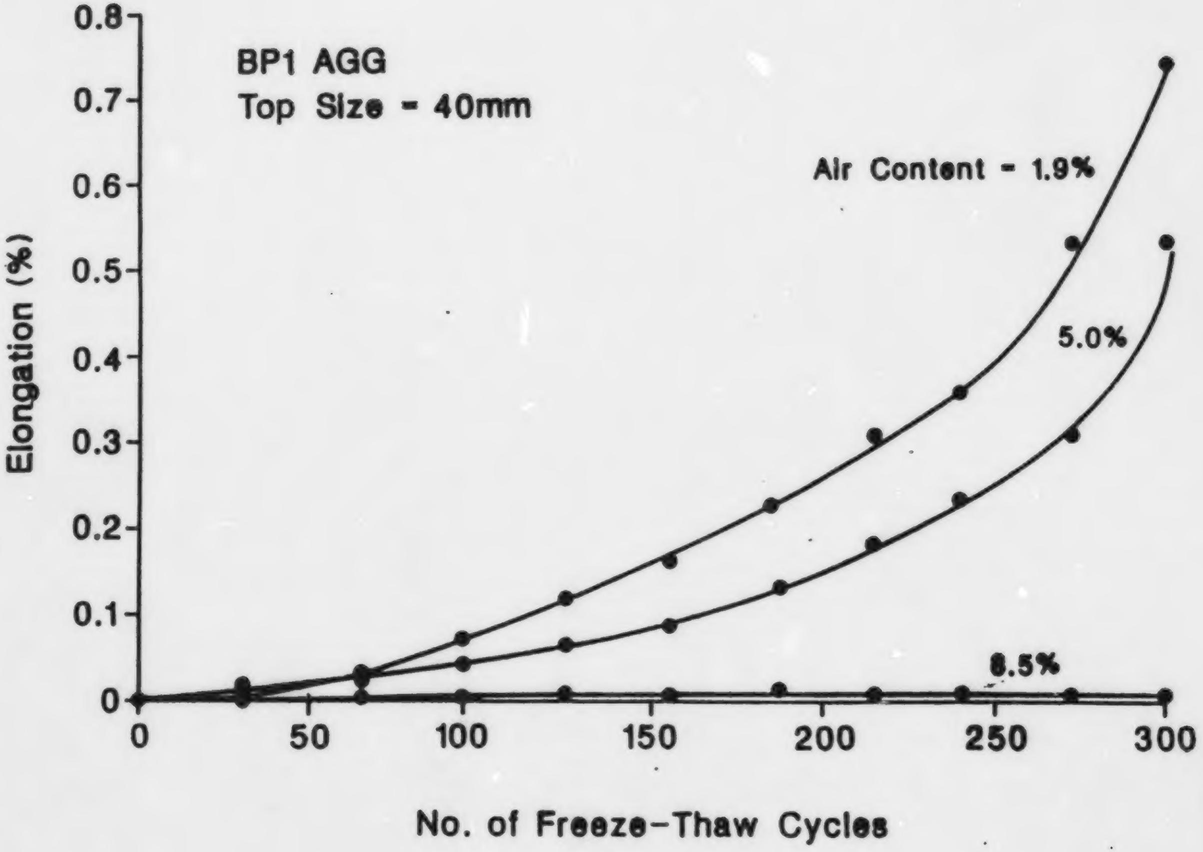


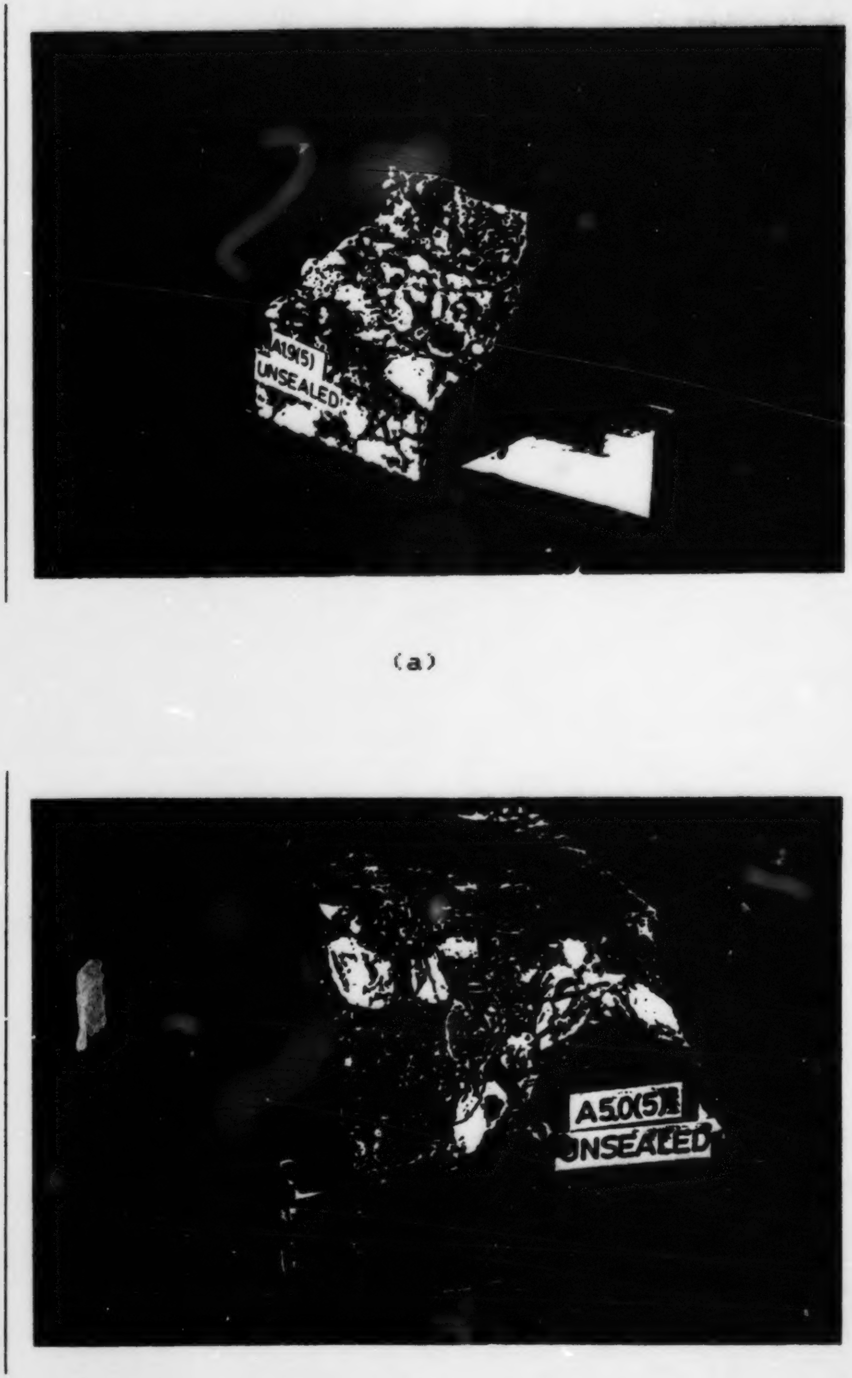
Figure 2.4 Results of fly ash substitution study (after Domaschuk et al., 1988)

were far in excess of the Traylor criterion, whereas the prisms having an air content of 8.5% were all well below the permissible elongation, despite the lower compressive strength of this mix. Thus, increasing the air content to 8.5% rendered the aggregate nonsusceptible to D-Cracking. Photographs of prisms with air contents of 1.9% and 5.0% which had failed before 300 freeze-thaw cycles are shown in Figure 2.5. The photographs show evidence of extensive coarse aggregate fracture as well as some failure at the aggregate/paste interface. There was also evidence of staining, as well as some very fine surface cracks, conditions that are characteristic of D-Cracking.

The results of this phase of the study suggest that air entrainment may be an effective and practical means of substantially improving the resistance to D-Cracking of concrete made with carbonate aggregates. A possible explanation for the improved resistance is that the entrained air bubbles act to control destructive forces produced by the diffusion process, as suggested by the Free Energy Difference theory, which was discussed in Section 1.3.2 of the report. However, a more comprehensive study is required to confirm this, and to establish the amounts of entrained air required to prevent D-Cracking from occurring.

2.2.4 Conclusion

A study of the effect of air entrainment on D-Cracking indicated that an air content of 8.5% prevented D-Cracking from occurring, whereas air contents of 1.9% and 5.0% did


A1.9(5)
UNSEALED
A5.0(5)
UNSEALED

(a)

(b)

Figure 2.5 Photographs of failed prisms: air content study
(a) prism A1.9, 272 cycles; (b) prism A5.0, 300 cycles

not. The concrete with an air content of 8.5% had a much higher slump, and a significantly lower 28-day compressive strength, than did the concrete with 1.9% and 5.0% air.

2.3 Effect of External Moisture (Domaschuk, Ganapathy and Triffo, 1988)

2.3.1 Aggregates, concrete, and test procedure

For this study, the prisms for the freeze-thaw tests were cast from the same batches of concrete that were prepared for the previous study. Consequently, the air contents of the prisms were 1.9%, 5.0% and 8.5%. The two studies were in fact carried out simultaneously. The properties of the fresh and hardened concrete are given in Table 2.5. Three prisms were cast from each batch and, after they had cured for 28 days, they were sealed with an epoxy coating and subjected to the freeze-thaw test.

2.3.2 Test results

The results of the test are shown in Figure 2.6. It is seen that 7 of the total of 9 prisms, underwent very little linear expansion. One exception was a prism from the 1.9% air content batch and the other was a prism from the 5.0% air content batch. The other two prisms from each of those same 2 batches performed as satisfactorily as did the 3 prisms from the third batch which had an air content of 8.5%. Thus, air content did not have any apparent influence on the performance of the prisms and the results can be interpreted solely on the basis of the availability of external moisture.

As indicated in Figure 2.6, the elongation of prism A.19(1) was minimal during the first 90 freeze-thaw cycles, then proceeded to increase exponentially. The rapid increase in elongation was very likely caused by a single large aggregate

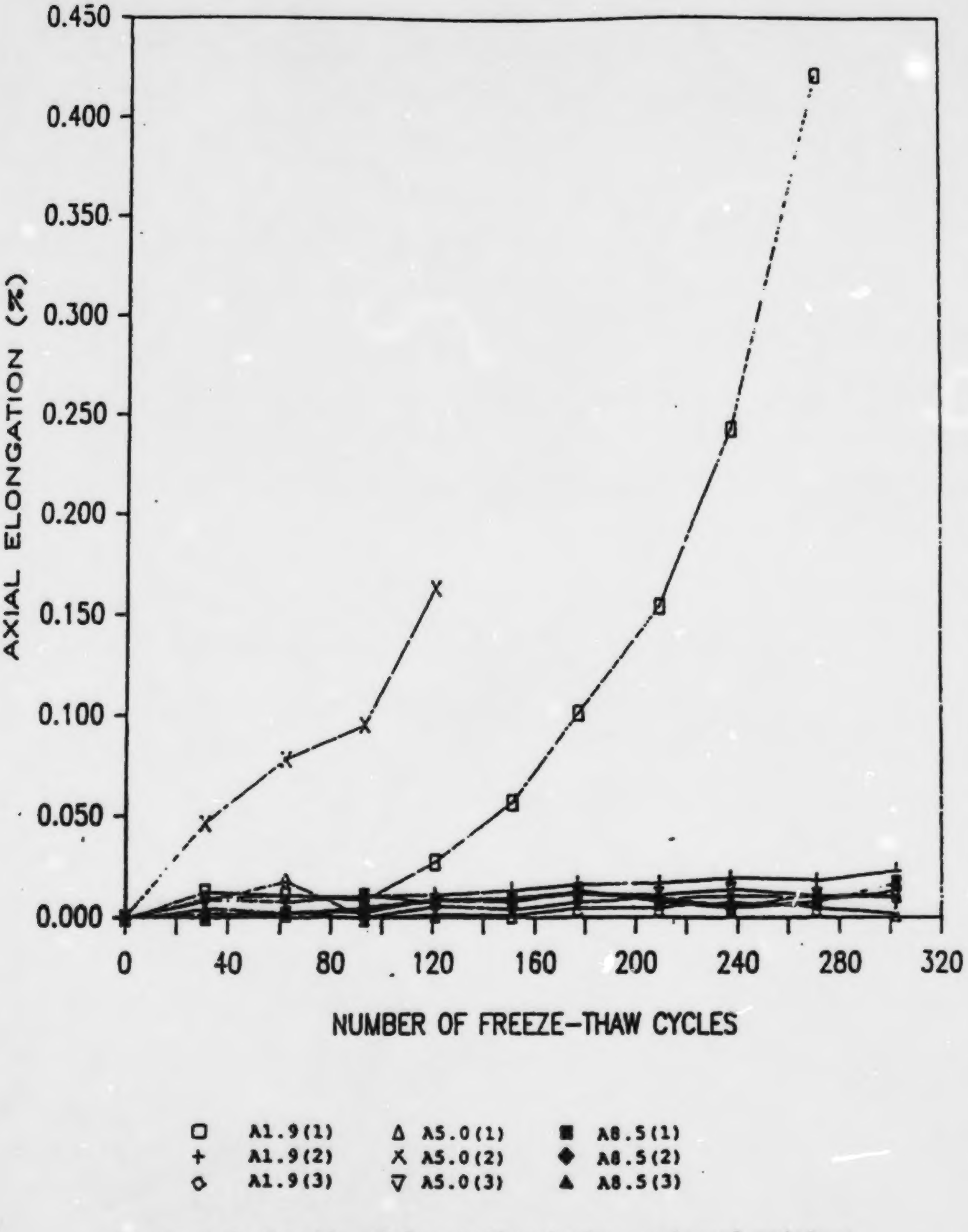
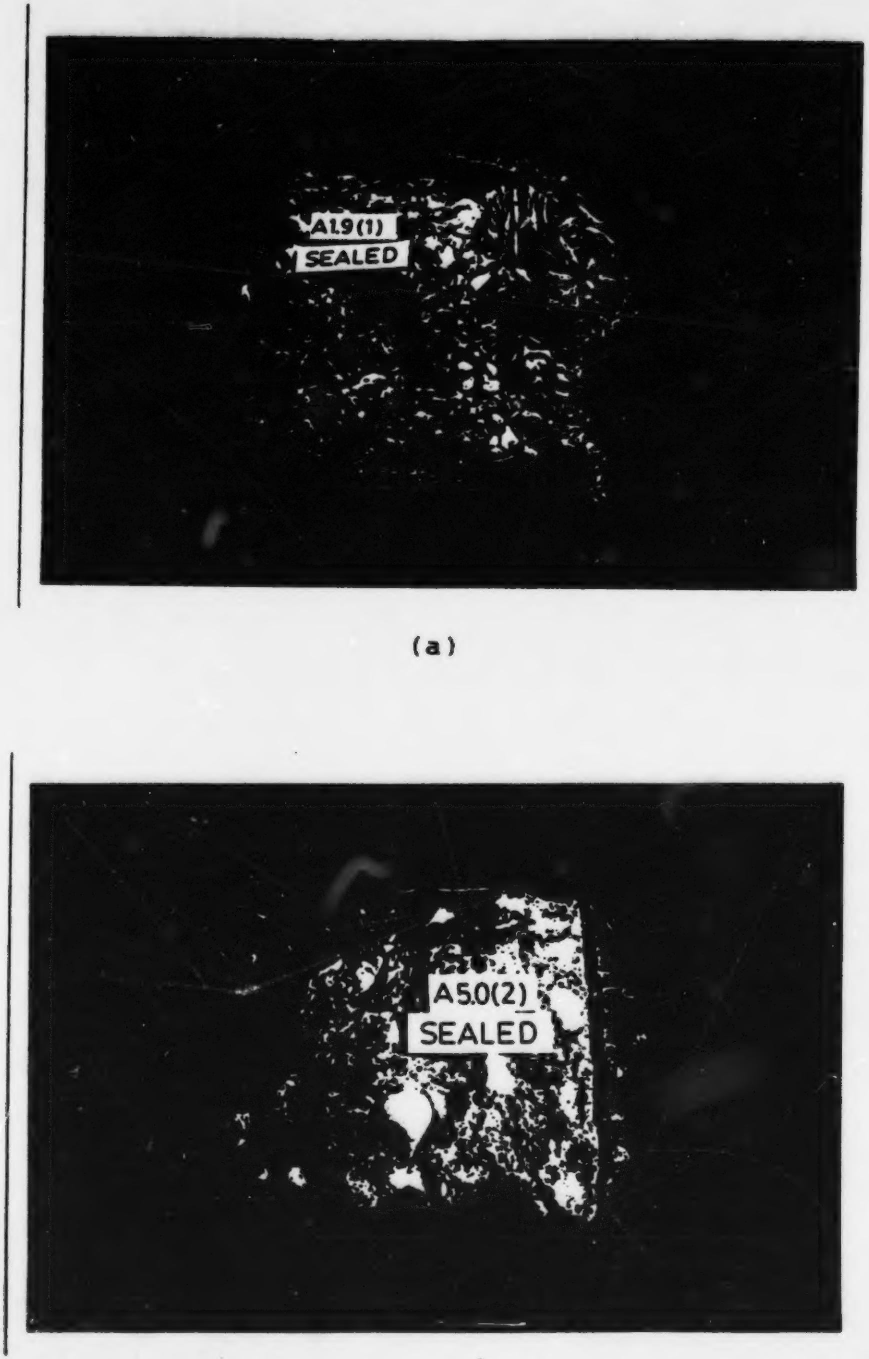


Figure 2.6 Results of freeze-thaw tests: external moisture study (after Domaschuk et al., 1988)

very near the surface of the prism. The aggregate can be seen in the upper right hand corner of the prism in Figure 2.7a. The photograph shows severe fracturing of this aggregate along bedding planes which are approximately parallel and about 3 mm apart. All other exposed large aggregates showed no evidence of fracturing. The fractured aggregate obviously had a very weak structure which was unable to withstand repeated cycles of freezing and thawing. Although a slight rupture of the epoxy coating directly over the aggregate was noticed at approximately 240 cycles, it would appear that the rupture was caused by the excessive expansion of the specimen in that area of the prism, rather than the rupture causing the expansion by providing access to external moisture.

In the case of prism A5.0(2), which also underwent excessive elongation, an examination of the prism after it had failed (after only 120 cycles) also revealed that the deterioration could be attributed to a single large aggregate located in the upper left hand side of the prism. (See Figure 2.7b). Here again the aggregate fractured along bedding planes which rendered the aggregate susceptible to fracturing. No other exposed aggregates showed evidence of fracturing.

Although the test results did not prove, unequivocally, that external moisture is a requirement for D-Cracking, the statistical evidence certainly supports the concept. However, the study showed that aggregates with an exceptionally weak structure do not require external moisture to fracture, even under a very limited number of freeze-thaw cycles.


A1.9(1)
SEALED
A5.0(2)
SEALED

(a)

(b)

Figure 2.7 Photographs of failed prisms: external moisture study (a) prism A1.9(1), 300 cycles; (b) prism A5.0(2), 186 cycles

2.3.3 Conclusion

In a study to determine if external moisture was an essential requirement for D-Cracking, 7 out of 9 concrete prisms, which had been sealed with an epoxy coating, performed satisfactorily, while 2 prisms underwent excessive expansion. The expansion in each of these cases was attributed to a single large aggregate with a very weak structure.

The study therefore suggests that, although external moisture plays an important role in the D-Cracking process, it is not an essential requirement when very weak aggregates are used.

2.4 Pore Structure Analyses of Durable and Nondurable Aggregates (Domaschuk and Garychuk, 1988)

2.4.1 Introduction

As mentioned in Chapter 1, the pore structure of an aggregate is thought to play an important role in the D-Cracking phenomenon. The pore structure includes: volume of pores, their surface area, pore size, pore shape, and size distribution.

Carbonate sedimentary rocks ranging in composition from pure limestone to dolomite have been identified as frost susceptible (Stark, 1976). The coarse carbonate aggregates identified as frost susceptible, had a pore-size range between 0.04 and 0.02 microns. Tests using ceramic samples of different porosities (Ravaglioli and Vecchi, 1973) implied that a high percentage of pores between 0.25 and 1.4 microns resulted in high frost susceptibility. Research on frost action in porous systems (Litvan, 1973) indicated that pores with radii in the range of 0.30 to .04 microns cause frost damage most often. Petroleum recovery investigations by Wardlaw (1976) using artificial and theoretical pore network models, suggested that the important aspects of pore systems were: pore to pore-throat ratio, throat to pore connectivity and the types and extent of random and non-random heterogeneities of the system.

Ultimate strength tests conducted on dry, low-porosity rocks with a wide range of texture and dolomite content (Hugman and Freidman, 1979) showed that grain size was the dominant intrinsic rock property that affected the ultimate strength in low porosity carbonate rocks. Limestones and dolomites composed entirely of micro-crystalline carbonate were stronger and more brittle than coarse-grained carbonates. Ultimate strength was found to increase with increasing dolomite content in similarly textured rocks, but not necessarily in differently textured rocks. Pure dolomites are generally stronger and more brittle than limestones.

Many different tests have been developed in an attempt to correlate a specific property or group of aggregate pore properties, to freeze-thaw durability. The pore-structure related properties that have been most commonly evaluated are: porosity, absorption, surface area, permeability, capillarity, pore-size distribution and petrographic description.

Total porosity has been determined by optical techniques such as microscopic examination of polished or thin sections, stereographic procedures of point counting, linear traverse and aerial measurement. The ASTM (C457) test includes the microscopic determination of the total porosity of the macropores but the micropore-structure remains undetected. Pycnometer methods using a gas such as helium or air, a pressure vessel, Boyle's law and an independent measure of bulk density, have been used to determine the effective porosity (Dolch, 1978). An average or equivalent pore-size can be approximated by measuring specific surface area, permeability, absorbtivity or capillarity. Calorimetric determinations of pore-size distribution have indicated a relationship between the pore-size, the triple-point temperature of water and the heat of solidification (Eyraud, 1973).

Absorption is a measure of effective porosity that expresses the volume of pores filled on attainment of equilibrium between the water pressure and the resistance to flow. Absorption measured in standard tests does not necessarily represent the total interconnected porosity volume as is commonly assumed. Complete saturation is rarely obtained under normal test conditions due to trapped air and shielded porosity. Two standard absorption tests are the ASTM C127 and C128 tests which measure the specific gravity of coarse and fine aggregates respectively. Other absorption tests include the Iowa Air Index test, the Mercury Intrusion test as well as diffusion tests.

Mercury intrusion porosimetry (MIP) is the most common method for pore-structure determination. However, there are two major drawbacks to the test. First, the pore diameters recorded are not necessarily the true pore diameters, but are the pore entry diameters of the pore network. Second, there exists "lost porosity", which is attributed to the inability of the mercury to invade the complete pore network of the aggregate (Alford and Rahman, 1981). The pore-size distribution calculations are therefore weighted towards the fine end of the pore-size range by the presence of these "ink bottle" pores which shield larger connected pores from lower pressure intrusion. Depressurization and second intrusion (Cebeci et al., 1978) was found to be helpful in distinguishing pores of uniform cross-section from "ink bottle" pores. This resulted in more comprehensive pore-structure information than could be obtained by conventional MIP tests.

The permeability of a porous media is the capacity for allowing continuous flow of a liquid or gas to take place through a system of continuous pores. The displacement of ions or gas molecules through a porous solid under the influence of a concentration gradient is called diffusion. Various permeating media such as water, nitrogen and air have been used to correlate permeability to D-Cracking incidence (Larson et al., 1964). These procedures assume that Darcy's Law is valid. The pore-structure dependent permeability coefficient is affected by the mode of mass transfer, either diffusion or capillarity. Permeability is also dependent on the effective porosity, saturation and capillarity. Correlations of permeability to freeze-thaw deterioration hve not been conclusive.

Aggregate and mortar capillarity have been measured and correlated to frost susceptibility (Larson et al., 1964). The idealized pore-structure model of bundles of different size capillary tubes differs substantially from the actual pore-structure. Therefore, no quantitative aggregate or pore structure measurements can be accurately obtained from these tests.

Petrographic examination of concrete aggregates is intended to provide a detailed description of the various pore related parameters such as mineralogy, surface texture, porosity, permeability and fabric. Common techniques frequently used in geotechnical petrographic analysis include the use of thin sections, hand lens, petrographic and stereographic microscopes, scratch tests, HCl treatment and carbonate staining. Less common are x-ray diffraction and scanning electron microscopy.

2.4.2 Pore-Casting and Image Analysis

Pore-casting is a well developed method of 2-dimensional pore structure examination of carbonate aggregates. The procedure combines vacuum evacuation followed by pressure application of a low viscosity resin that impregnates the interconnected pores of a disk-shaped aggregate specimen. A coloured or fluorescent dye is added to the resin for optical recognition. The specimen is ground and polished and then etched with dilute HCl and HF acid to remove the carbonate host, leaving a relief profile of the pore structure. Pore sizes down to 0.1 micron have been impregnated in this fashion (Wardlaw, 1976). There are various problems that can arise during sample preparation of pore-casts. If the material contains hard-brittle particles, they may be torn out and possible re-imbedded in a different part of the surface, during polishing. If too much pressure is applied during polishing, the surface may become badly distorted. If the specimen contains constituents that differ appreciably in hardness, the removal of the softer grains produces a relief profile. Other limitations of pore-casting are that isolated porosity may not be intruded and high pressure of resin intrusion may produce pore-cast collapse.

In simple terms, image analysis is a facet of stereology that quantitatively evaluates the statistical and geometrical properties of 2-dimensional images. Image acquisition techniques that have been used for pore-structure analysis are optical and SEM microscopy, photographs, negatives, reflected light from planar sections or transmitted light through thin sections. Image Analysis Procedures (IAP) and Petrographic Image Analysis techniques have been well developed and extensively used in pore-complex research (Rink, 1973; Ehrlich et al., 1984).

In the study at the University of Manitoba, an erosion-dilation technique developed by Ehrlich et al., was used for the analysis of image features. The procedure is as follows: Firstly, detectable images are obtained using a video-camera system integrated with an analog/digital converter that electronically digitizes these images into a matrix of binary data. Each picture point in the image matrix, referred to as a "pixel", is defined by three values: two spatial coordinates (X,Y) and a "grey level" intensity value. A computer is used to process these digital images using various complex pattern recognition algorithms and data manipulation computer programs.

One erosion cycle performed on an individual pore is the removal of the outermost layer of pixels from the pore segment. Dilation is the reverse of erosion, where a layer of pixels is added to the object. Dilation after an erosion does not necessarily restore the object to the original shape. As erosion-dilation cycles proceed, pores of progressively larger size are lost and the surviving elements are of simpler geometry. This continues until all pixels have been removed by erosion. These procedures are illustrated in Figure 2.8. Image A and A' represent the original pore-structure. The effect of erosion is that it strips off the outer layers of "pixels" of the image illustrated by image B. Dilation is the opposite of erosion in that it adds "pixels" to the defined image. Image B undergoes dilation and produces image B'. The image is further eroded producing image C and subsequent dilation produces image C'. It is noted that the large pores have smoother surfaces and the smaller pores are absent.

Pattern recognition/classification algorithms have been developed that consist of monitoring the proportion of the porosity that is "lost" under progressively more severe cycles of erosion-dilation. Each cycle begins with the original image of a single pore-segment selected from the field of view under investigation. The pore-segment undergoes erosion-dilation cycles until at some point the entire pore disappears and the dilation has nothing to restore. The portion of the pore that exists just before this, is referred to as the "pore porosity". The proportion of the rest of the porosity of that pore that is monitored with each cycle is referred to as "roughness porosity". Analyzing all of the pores in a given field of view in this manner results in a frequency distribution of the proportions of the total porosity lost in each cycle, the total being 100%. These frequency distributions have been termed "pore-complex spectra". An example of "pore complex spectra" for a given image is illustrated in Figure 2.9. The porosity has been divided into size and roughness distribution.

The description of the pore-structure involves the determination of five independent stereometric parameters; size, shape, particle or pore density, distribution and orientation. Features and characteristic shapes commonly used in image

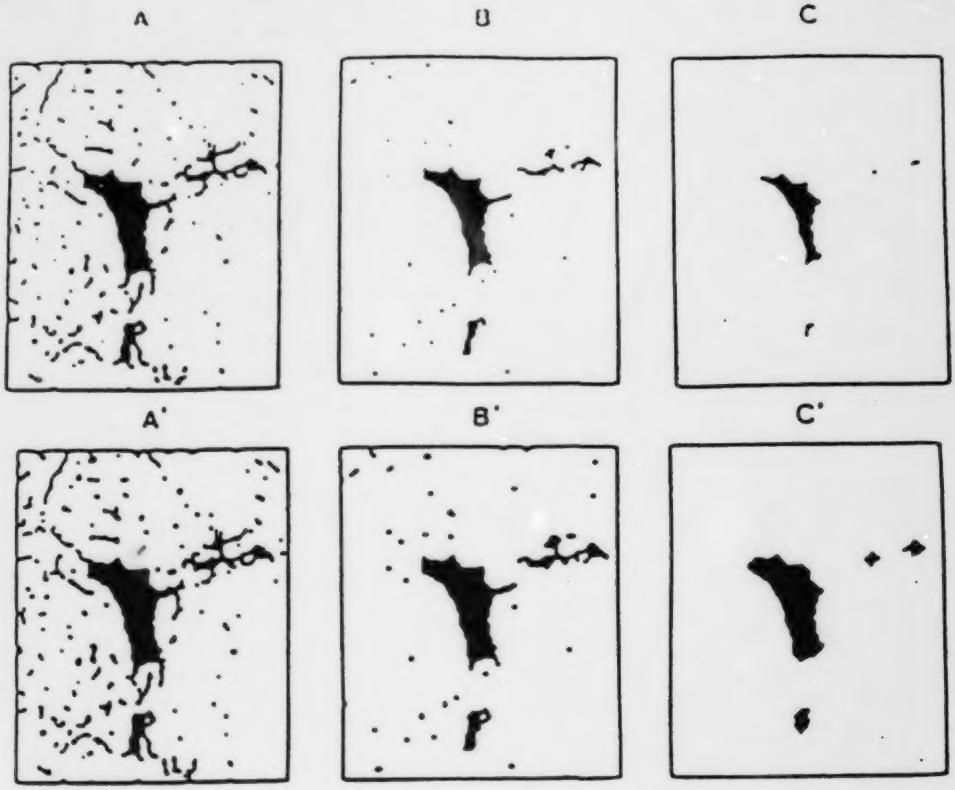


Figure 2.8 Erosion and dilation cycles (Ehrlich et al., 1984)

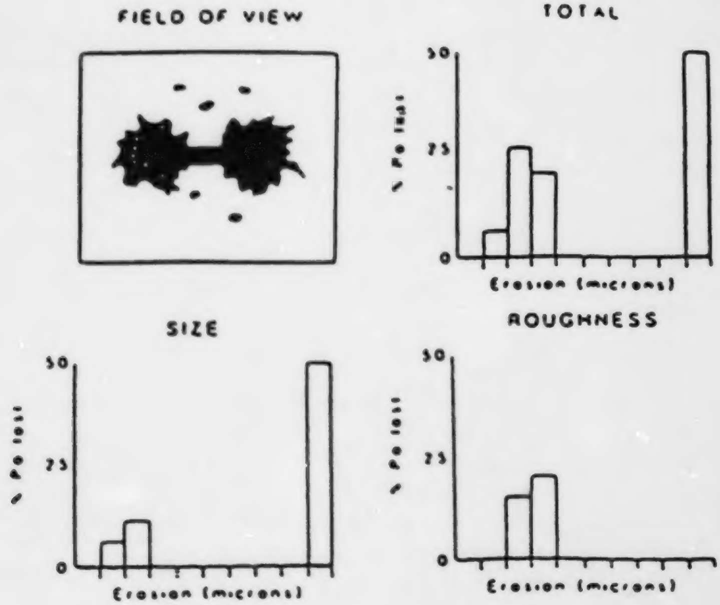


Figure 2.9 Separation of total porosity into size and roughness (Ehrlich et al., 1984)

analysis have been identified by certain form functions. Image analysis procedures have been used to separate these individual section figures on the basis of grey-level recognition, count them and calculate a host of size and shape parameters and statistical distributions. Theoretical analysis by Martin (1973) demonstrated that: (a) the volumetric porosity of a specimen can be expected to equal the aerial porosity; (b) the volumetric specific surface can be expected to equal the aerial specific surface, assuming the spatial pore distribution in the rock is statistically homogeneous and isotropic.

Comparisons have been made between rock properties deduced from image analysis with those determined by some other physical tests. In a study of sandstone samples (Rink, 1973; Dullien, 1973), good agreement was found in the case of porosity and mean hydraulic radius; poor agreement in the case of specific surface and capillarity; and varying degrees of agreement for permeability. The limitations of both image and physical test methods must be kept in mind when comparing results.

2.4.3 Laboratory Investigation

The object of the laboratory study was to apply basic petrographic analysis techniques for micro-porosity analysis to several carbonate rock aggregates of varying D-Cracking susceptibility, using a LEITZ TAS PLUS image analyzer.

Specimen preparation procedures for image analysis involved pore-casting, i.e. pressure-injecting a blue-dyed epoxy resin compound into the inter-connected pore space of the rock microstructure. This was followed by grinding and polishing. Microscopic detections of the blue-dyed pore segments, with a resolution down to 0.1 microns, were obtained from these pore-casts.

A software program was developed for the LEITZ image analyzer to carry out the microscopic pore segment analysis. This program analyzed the Total Optical Porosity (TOP) of the samples and quantified it into porosity frequency distributions called pore-complex spectra. Pore-complex spectra for several of the aggregate samples are presented and analyzed. Details of the samples, specimen preparation, image analysis techniques and the results are provided by Garychuk (1986, 1987). They are summarized in the sections that follow.

2.4.3.1 Aggregate Samples

One sample group consisted of crushed carbonate quarry aggregates provided by the Illinois Department of Transport Research. The results of freeze-thaw tests carried out in accordance with the ASTM C-666 Method B test for aggregate durability, are listed in Table 2.6. The numbers in this table represent the elongation of the concrete test prisms after 300 test cycles. Values in excess of 0.06% generally indicate that the aggregate is susceptible to D-Cracking. Thus, the samples represented a wide range of susceptibility, with elongations varying from 0.005 to 0.191%. These samples were used primarily for the development and refinement of pore-casting and image analysis techniques and hereafter are referred to as control samples.

Table 2.6: Freeze Thaw Test Results (Illinois DOT)

CODE	PRODUCER	ELONGATION (%)
FT4-103	Specification Stone, Pana	0.078
FT4-106	Vulcan Materials, Lehigh	0.005
FT4-108	Columbia Quarry, Ullin	0.191
FT4-109	Western Ill. Stone, Quincy	0.120
FT5-18C	Reed Crushed Stone	0.076
FT5-21B	Vulcan Materials, McCook	0.015

A second sample group was chosen from the coarse carbonate fraction of local aggregates which had been used in the studies at the University of Manitoba, previously described. As indicated earlier, the BP-2 aggregate (Birds Hill, 40 mm top size) was extremely susceptible to D-Cracking, and all other aggregates performed satisfactorily.

Aggregate particles for image analysis were taken from the test prisms that had been used in the study, described in Section 2.1. The prisms were sectioned into 3 mm slices and coarse aggregate particles were recovered from the slices. Particles that exhibited signs of cracking were assumed to be non-durable. Individual particles were selected on the criterion of being either durable or non-durable. The non-durable aggregate particles were taken from the prisms that had failed after 150-250 cycles. In addition, two non-durable aggregate particles were recovered from the S2 prisms which had revealed some deterioration. All of the durable aggregate particles were recovered from the S1 prisms which had showed no evidence of deterioration after being subjected to 300 freeze-thaw cycles.

2.4.3.2 Pore-Casting

Pore-casts were obtained using a rock impregnation technique developed at the University of Calgary (McKellar and Wardlaw, 1981). The basic procedure was as follows:

1. A small fragment (diameter $<$ 20 mm) of each aggregate particle to be analyzed was washed thoroughly in an ultrasonic bath, air dried, then placed in a 30 ml plastic micro-beaker.
2. The epoxy resin-hardener compound was then mixed according to specifications with a small quantity of blue dye added, then was poured into the beakers containing the specimen fragments.
3. The beakers were then placed in a vacuum bell jar and were subjected to a vacuum for about 15 minutes to evacuate the pores.
4. The bell jar was allowed to come to atmospheric pressure and the beakers were removed and placed into the pressure bomb.
5. The samples were subjected to a vacuum pressure for an additional 20-30 minutes, the vacuum was

turned off, and pressure from a nitrogen bottle was increased gradually up to 13,800 kPa. This pressure was maintained for about 20-30 min.

6. After the pressure impregnation period, the pressure was gradually reduced to atmospheric, and the samples were placed in a fume hood to cure for 16 to 24 hours.

7. The samples were then ground down, sanded, and polished to expose the rock surface.

Both red and blue dyes were used in the pore-casting procedures. The blue dye was found to provide greater optical recognition characteristics when viewed under the LEITZ microscope. Carbonate rocks by nature are very soft and susceptible to scratching during polishing procedures. Care was taken during polishing to minimize scratching. Polishing down to 0.25 microns was required to ensure a quality finish on the pore-cast specimens. Figure 2.10 shows a pore-cast plug after grinding, sanding and polishing.

2.4.3.3 Computer Analysis

The LEITZ TAS PLUS image analysis computer is a fully programmable computer system developed for the automatic processing and quantifying of optical images. For the purpose of this study, a program was synthesized to facilitate microporosity analysis of polished sections of blue-dyed carbonate rock pore-casts. The algorithm for this program was adapted from Petrographic Image Analysis (PIA) techniques developed at the University of South Carolina (Ehrlich et al., 1984).

The pore-complex analysis of the pore-casts was carried out using reflected light microscopy. The micro-porosity analysis program calculated the total optical porosity (TOP) of the interconnected pore space of the rock micro-structure. Each bit plane was represented by ten randomly spaced fields of view. An example bit plane image of a pore-cast viewed on the LEITZ computer microscope is illustrated in Figure 2.11. This image consists of a 512 x 512 pixel array. The field of view in this image is 182.5 microns which corresponds to a magnification of 1280X. This magnification allowed for the detection of microporosity down to 0.3 microns which corresponds to 1 pixel. However, pore geometry can only be determined when a pore is expressed over a minimum of 4 to 8 pixels. At higher magnifications, resolution of larger pores within a field of view became very distorted and undetectable. Focusing, lighting and detection problems also became appreciable at these magnifications, making them unacceptable. For these reasons a magnification of 1280X was found to be optimum for the investigation of the microstructure of these particular carbonate rock pore-casts. All samples were analyzed at the same magnification to facilitate qualitative and quantitative comparisons.

A sample "pore-complex spectra" produced by this program, extracted from the image in Figure 2.11, is given in Figure 2.12. This printout includes the number of frames analyzed, total optical porosity (TOP) and the total pore roughness and number of frequency distributions.

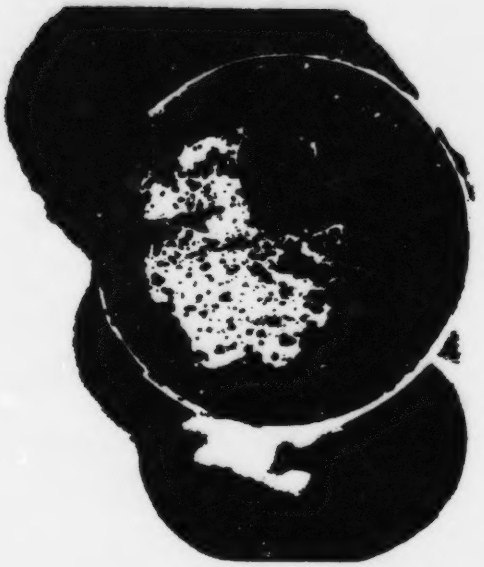
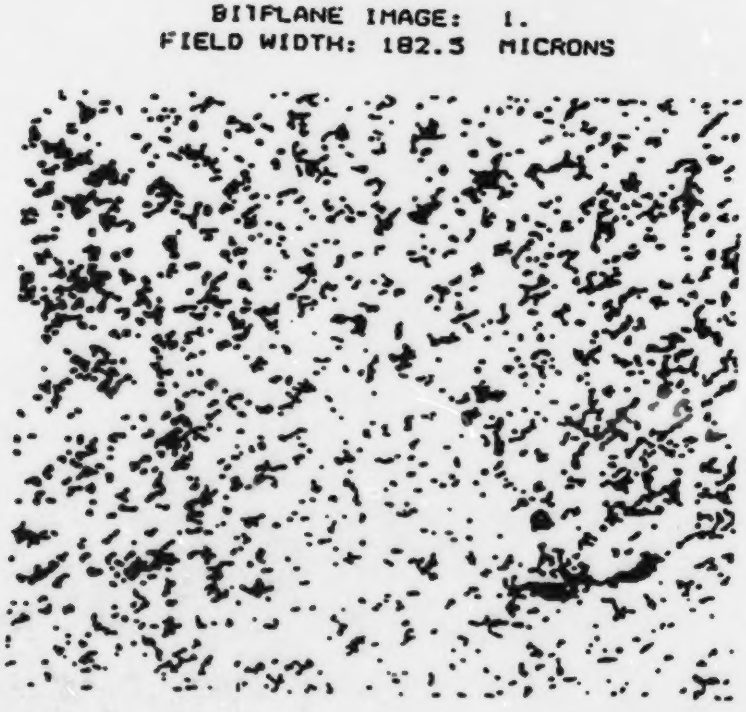


Figure 2.10 Pore-cast sample (diam. = 25 mm)



BITPLANE IMAGE: 1.
FIELD WIDTH: 182.5 MICRONS

Figure 2.11 Computer image of pore-cast

```
PORE STRUCTURE ANALYSIS OF CARBONATE ROCK [illegible] PORE CASTS
POROSITY ANALYSIS
PORE COMPLEX SPECTRA

SAMPLE #:  S1-10
# OF FRAMES:  10.

TOTAL OPTICAL POROSITY
   >>>19.39% <<<

                          TOTAL POROSITY

% POROSITY      E/D CYCLES
  53.3100        1.00000      +----------------------------------------------------*
  35.1449        2.00000      +----------------------------------*
  7.73346        3.00000      +-------*
  1.56172        4.00000      +-*
 0.537981        5.00000      *
 0.894250E-01    6.00000      *
 0.000000        7.00000      *

                          PORE POROSITY

% POROSITY      E/D CYCLES
  30.3901        1.00000      +-----------------------------*
  28.8787        2.00000      +----------------------------*
  6.87355        3.00000      +------*
  1.45727        4.00000      *
 0.514372        5.00000      *
 0.894250E-01    6.00000      *
 0.000000        7.00000      *

                          ROUGHNESS POROSITY

% POROSITY      E/D CYCLES
  22.9279        1.00000      +----------------------*
  6.26619        2.00000      +-----*
 0.859910        3.00000      *
 0.104448        4.00000      *
 0.2360828E-01   5.00000      *
 0.000000        6.00000      *
 0.000000        7.00000      *

                          NUMBER FREQUENCY

# OF PORES      E/D CYCLES
  9675.00        1.00000      +----------------------------------------------------*
  2587.00        2.00000      +-------------*
  311.000        3.00000      +-*
  40.0000        4.00000      *
  8.00000        5.00000      *
  1.00000        6.00000      *
 0.000000        7.00000      *
```

Figure 2.12 Pore-complex spectra

2.4.3.4 Sample Analysis

Twenty-six samples were analyzed using the "Pore-Complex Spectra" program on the LEITZ TAS PLUS image analyzer. The Total Optical Porosity (TOP) for each of the specimens is illustrated in the bar graph in Figure 2.13. The sample groups have been separated and plotted in ascending order of porosity. This plot illustrates that the non-durable specimens exist in a narrower range of TOP than that of the durable specimens. The TOP ranged from 15.1% to 23.4% for the non-durable specimens, 5.2% to 22.4% for the durable specimens and 3.2% to 22.5% for the control specimens. One of the durable specimens recorded a porosity of 39.0% but it was felt this high value was an error attributable to the algorithm. This particular carbonate rock type is a special case and its occurrence is rare. For these reasons this specimen was eliminated from subsequent analysis.

Figures 2.14 to 2.16 further illustrate the TOP of the individual sample groups separated into "pore" and "roughness" porosity. Analysis of the proportions of pore and roughness porosities did not indicate any correlations with D-Cracking susceptibility.

The first erosion-dilation cycle (1 E-D) eliminates the micro pores. Figures 2.17 and 2.18 illustrate the percentages of the 1 E-D "pore porosity" of the non-durable and durable sample groups respectively. Analysis of these data indicates that the non-durable sample group, with the exception of one sample, had a higher absolute percentage of 1 E-D pore porosity ($<$ 0.3 micron pores) than did the durable sample group. This was consistent with the findings of others.

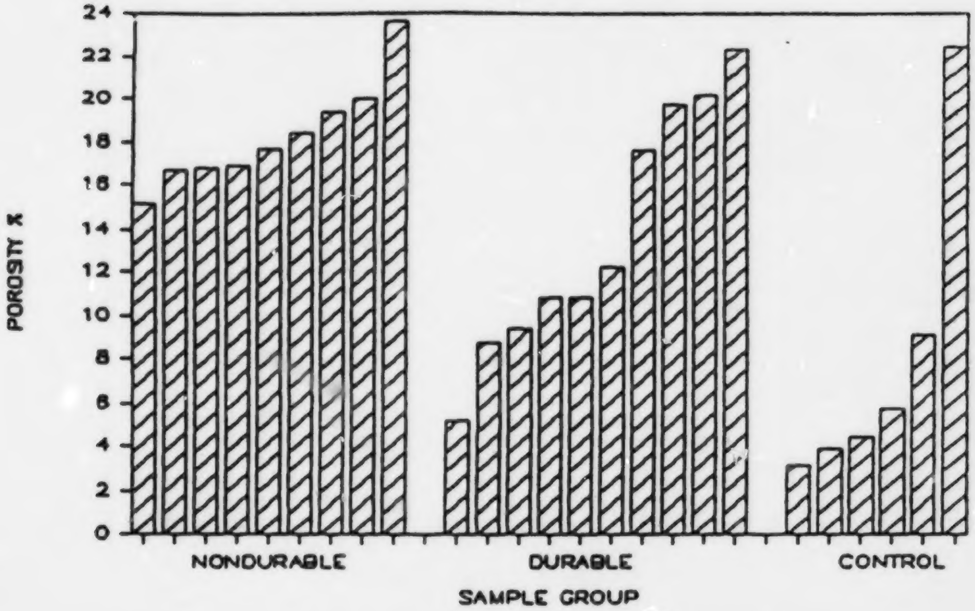


Figure 2.13 Total optical porosity: all samples

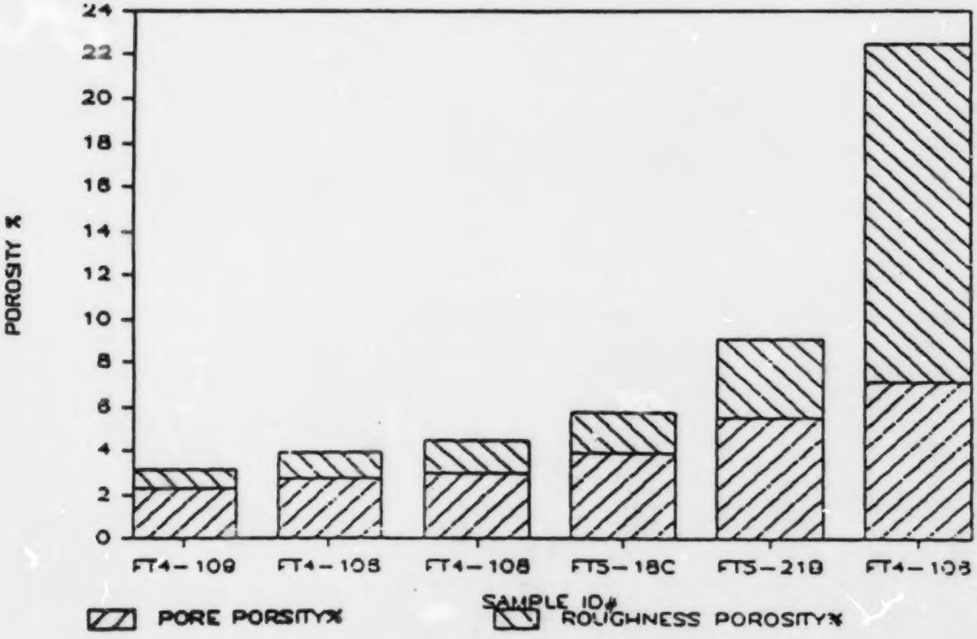


Figure 2.14 Total optical porosity: control sample

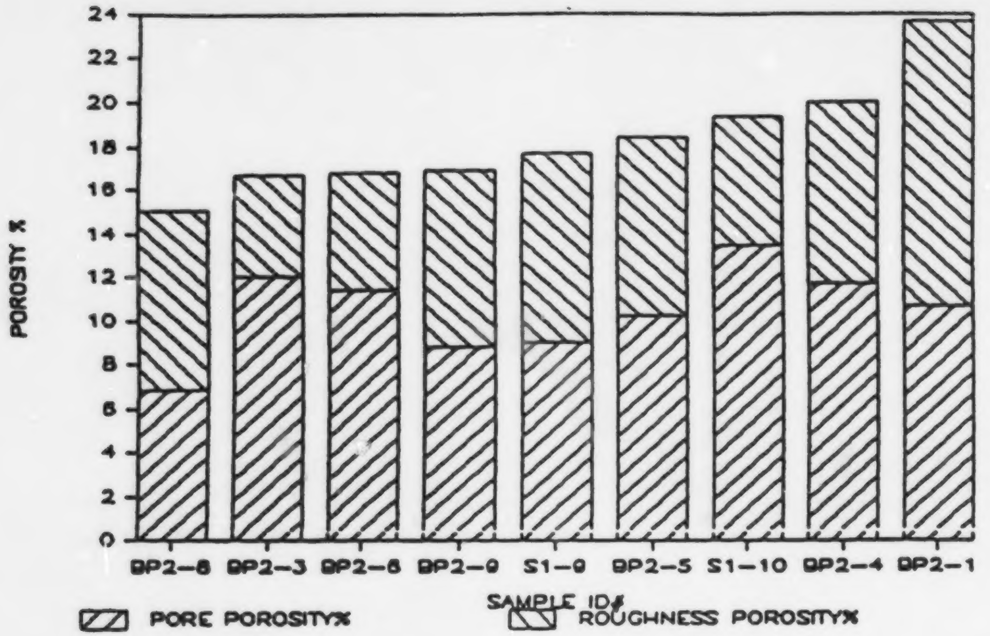


Figure 2.15 ·Total optical porosity: non-durable sample

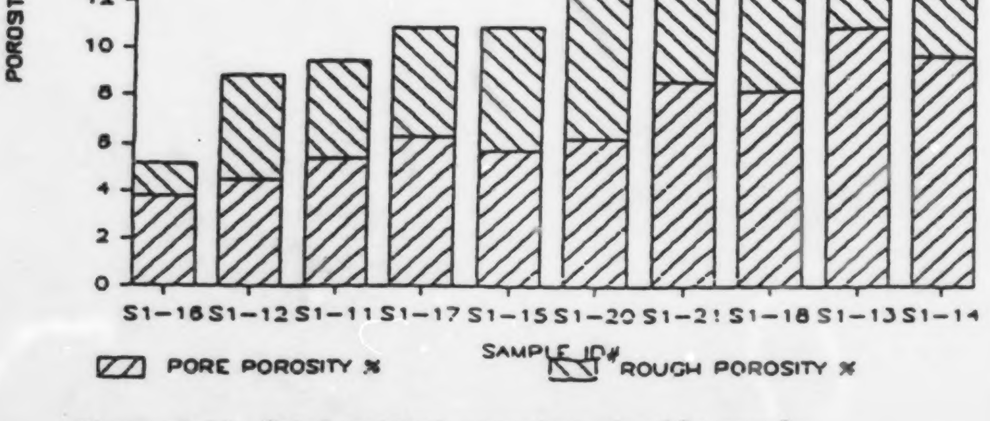


Figure 2.16 Total optical porosity: durable sample

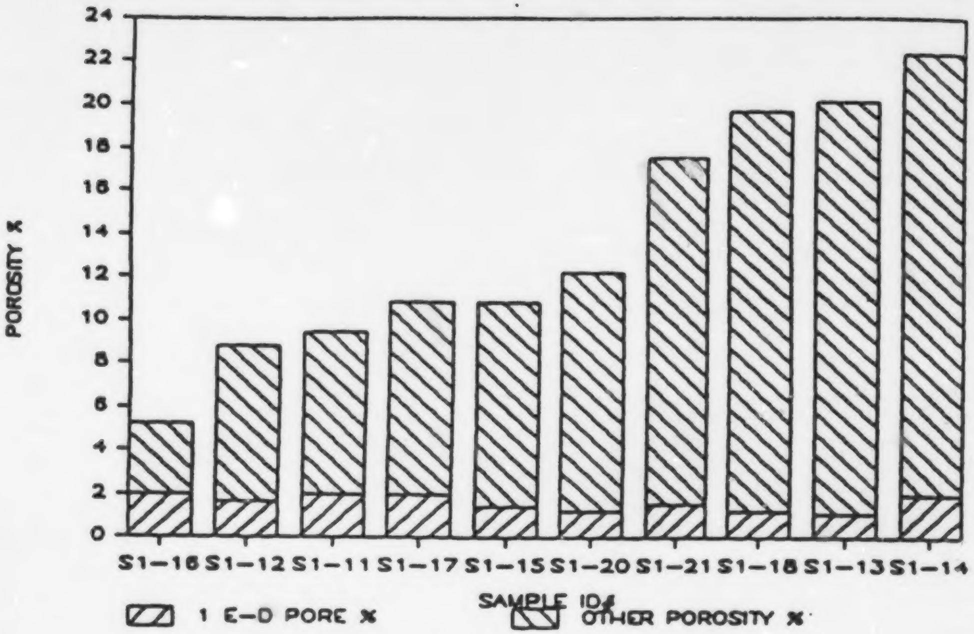


Figure 2.17 1 E-D pore porosity: durable sample

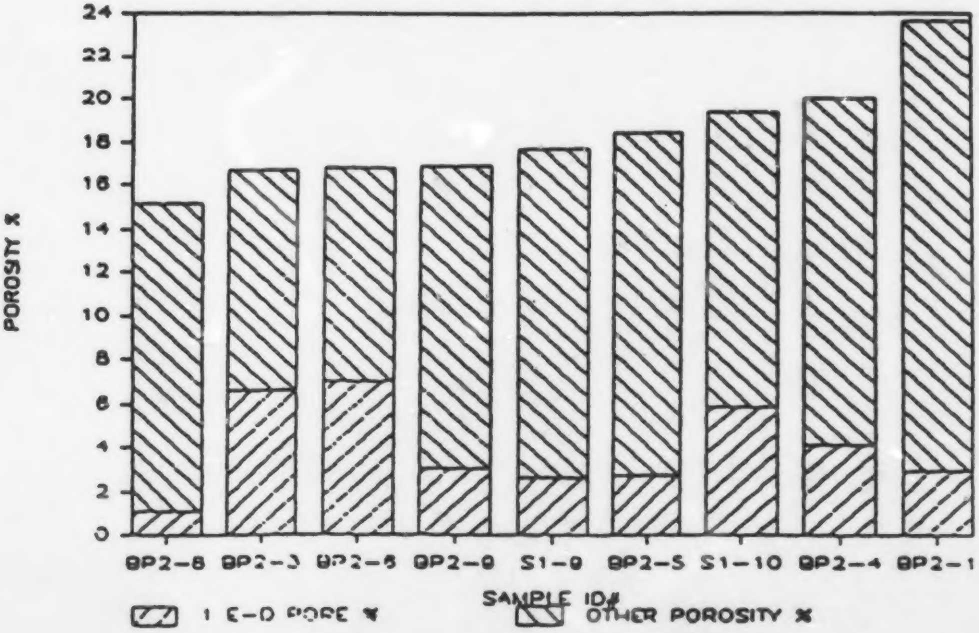


Figure 2.18 1 E-D pore porosity: non-durable sample

The algorithm used to calculate the pore porosities tended to overestimate the roughness porosity portion of pore segments. This resulted in the pore-complex spectra being weighted towards the roughness porosity proportion, especially when analyzing larger pores. Occurrences such as pore separation (when a pore splits into two or more segments) also resulted in significant overestimation of roughness porosity. When a portion of a split segment disappeared, it was registered as roughness porosity rather than part of the pore porosity.

The very large computer memory space allotted for program and parameter storage, limited the capacity of LEITZ TAS PLUS for the development of Petrographic Image Analysis procedures. This limitation was subsequently overcome by computer interfacing, involving the direct transfer of image data to a microcomputer for data processing.

2.4.3.5 Observations and Conclusions

Twenty-six specimens taken from aggregates of varying susceptibility to D-Cracking were analyzed using a simplified "Pore-Complex Spectra" program developed for the LEITZ computer system.

Total Optical Porosity (TOP) ranged from 15.1% to 23.4% for the non-durable specimens and from 5.2% to 22.4% for the durable specimens. This data indicated that the non-durable aggregates tested in this study existed in a narrower range of TOP than did the durable aggregates.

The non-durable aggregates exhibited narrower pore-size ranges than those of the durable specimens.

The non-durable specimens had a higher absolute percentage of 1 D-D pore porosity (< 0.3 micron pores) than that of the durable specimens.

Correlations of the "pore" and "roughness" porosity to D-Cracking susceptibility was inconclusive.

The findings of this preliminary study indicated that a relationship between parameters obtained through PIA techniques and the D-Cracking susceptibility of carbonate aggregates may exist. A much more comprehensive study is required.

Some additional work on developing petrographic image analysis techniques was subsequently carried out at the University of Manitoba by Harvey (1987). He investigated the use of fluorescent dyes to improve the quality of pore cast images required for pore structure analysis. He also extended Garychuk's study of the correlation between pore properties as determined by image analysis and D-Cracking susceptibility. He found that, by using the "maximum entropy method", the frequency distribution of pore area provided correlations were statistically significant. However, the extent of the investigation was too limited to be conclusive.

Garychuk (1987) developed a menu-driven software program for processing the large data base generated by the image analyzer. The program uses the Chi-squared test to determine the statistical significance of measured parameters compared to the parent (combined) sample data base. Using the "maximum entropy method", the program may be used to correlate the image based parameters with the freeze-thaw durability of the aggregate.

2.5 Petrographic Studies of Manitoba Aggregates

2.5.1 University of Manitoba Study

2.5.1.1 X-Ray Diffraction (Garychuk, 1986)

For comparative purposes, aggregates known to be susceptible to D-Cracking (Birds Hill, BP2) and nonsusceptible aggregate samples (Ritcher, S1) were selected for x-ray diffraction. The samples were taken from the concrete prisms that had been used in the freeze-thaw tests.

A small portion of each aggregate was ground to a fine powder and spread over a glass slide. The slides were analyzed at 40 KV and 40 MA with nickel-filtered Cu K-alpha radiation on a Philips model PWWW1729 x-ray diffractometer. A medium speed (6 degrees 2-theta/min) was used and each sample was scanned 3 times from 28 to 32, 2-theta. This permitted average intensity measurements of calcite (3.03 Å) and dolomite (2.88 Å) reflections. The percent dolomite by weight was calculated using the ratio of (dolomite) to (dolomite + calcite) and the standard calibration curve shown in Figure 2.19. The results of the x-ray diffraction analysis are given in Table 2.7.

Carbonate rock-types may be classified according to the calcite-dolomite contents as shown in Table 2.8. According to the x-ray diffraction data, the rock types of the susceptible and nonsusceptible aggregates are given in Table 2.9. From Table 2.9 it is seen that there was not a very significant difference in the calcite/dolomite composition of the susceptible and nonsusceptible carbonate aggregates. Thus, there was no apparent correlation between the mineralogical composition and D-Cracking susceptibility, for the carbonate aggregates analyzed.

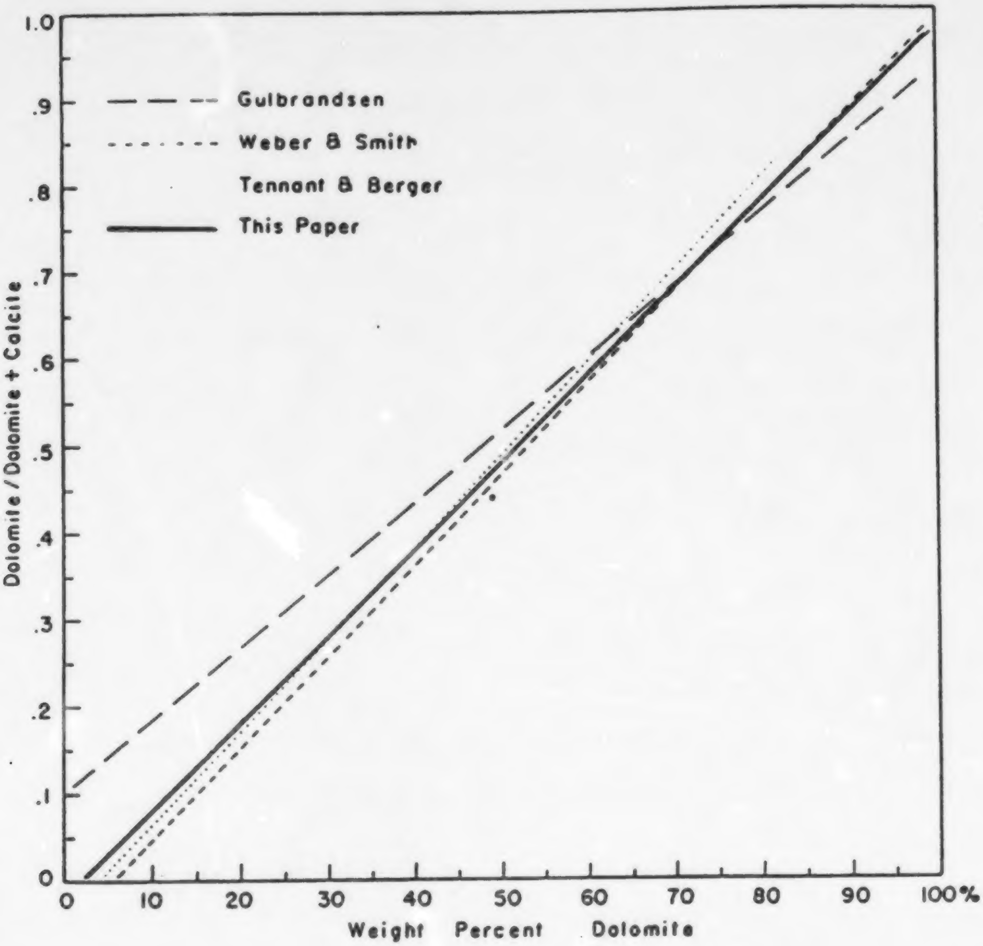


Figure 2.19 Weight Percent Dolomite Calibration Curve (Royse et.al. 1971)

Table 2.7: X-Ray Diffraction Data

SAMPLE NO.	DOLOMITE INTENSITY	CALCITE INTENSITY	(Do) (Do + Ca)	DOLOMITE WEIGHT(%)
Ritcher (nonsusceptible)				
S1-11	13061	152	0.99	100
S1-12	15668	321	0.98	99
S1-13	14168	0	1.00	100
S1-14	304	8311	0.04	6
S1-15	8908	217	0.98	99
S1-16	4437	3638	0.55	57
S1-17	13454	0	1.00	100
S1-18	13369	210	0.98	99
S1-19	4102	216	0.95	98
S1-20	1979	7410	0.21	23
S1-21	12975	149	0.99	100
Birds Hill (susceptible)				
BP2-1	9434	212	0.97	99
BP2-3	12093	97	0.99	100
BP2-4	13642	108	0.99	100
BP2-5	12380	164	0.99	100
BP2-6	4197	3322	0.56	58
BP2-7	8541	2807	0.75	78
BP2-8	13948	78	0.99	100

Table 2.8: Classification of Carbonate Rocks

% DOLOMITE	% CALCITE	ROCK CLASSIFICATION
100 - 90	0 - 10	Dolomite
90 - 50	10 - 50	Calcitic Dolomite 10
10 - 50	50 - 90	Dolomitic Limestone
0 - 10	90 - 100	Limestone

Table 2.9: Classification of Aggregates by X-Ray Diffraction

CLASSIFICATION	% SUSCEPTIBLE	% NONSUSCEPTIBLE
Dolomite	67	73
Calcitic Dolomite	11	9
Dolomitic Limestone	22	9
Limestone	0	9

2.5.1.2 Scanning Electron Microscope (SEM), (Garychuk, 1986)

Small fragments taken from the same aggregate particles that were used in the x-ray analysis, were examined with a Cambridge Sterioscan Mark II scanning electron microscope. The fragments, approximately 6 mm in diameter, were washed in an ultrasonic bath, were then mounted on sample holders, and a thin film (100 Å) of gold was applied with a Balzer sputter-coater in preparation for scanning. Samples of SEM photos are shown in Figure 2.20. The report on the SEM studies is given in Appendix A.

Generally, most of the nonsusceptible fragments indicated a relatively large pore-size range. This may have been very significant in providing the aggregates with a relatively high resistance to frost action. The inter-granular porosity was estimated to commonly range in the order of 2 to 5%.

The susceptible fragments generally had a very narrow pore-size range, a characteristic of aggregates with low

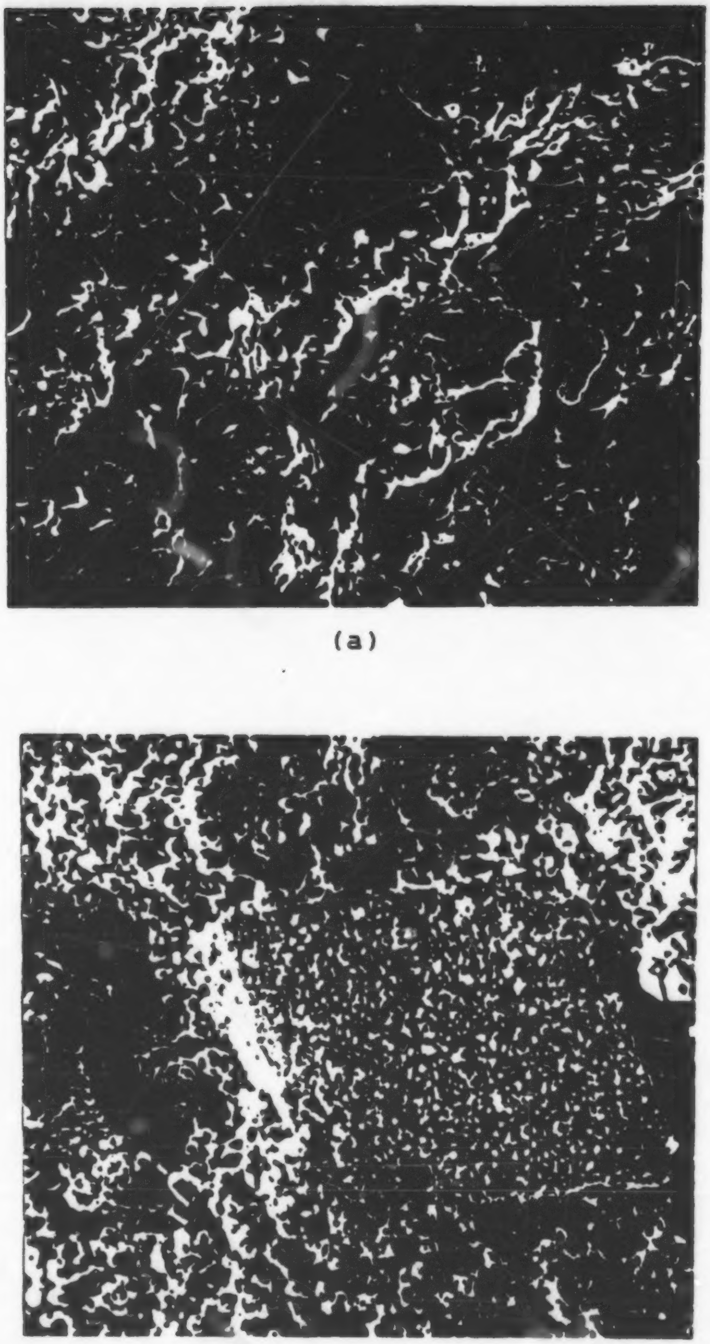
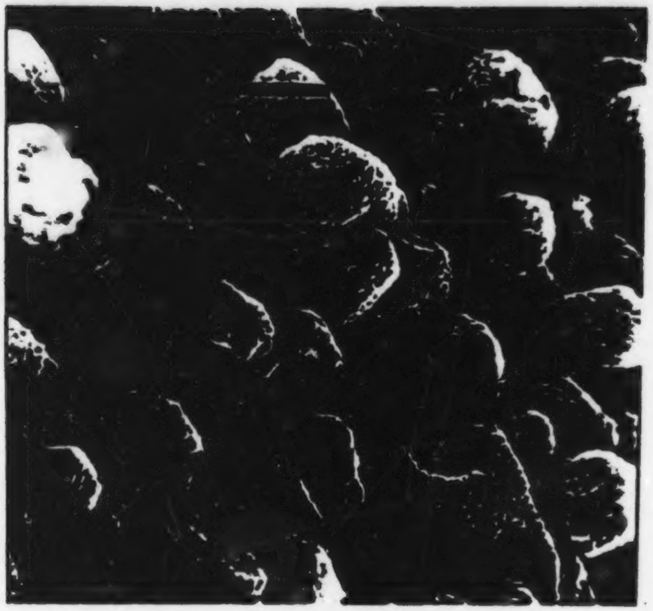
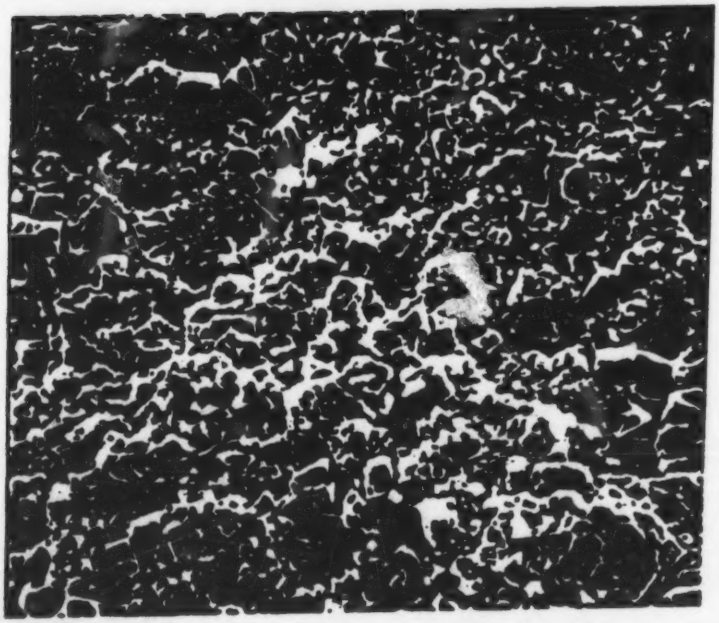


Figure 2.20 Samples of SEM photos: Durable Aggregate (a) Limestone; (b) Silicified Dolomite



(c)



(d)

Figure 2.20 Samples of SEM photos: Nondurable Aggregate
(c) Limestone; (d) Dolomite

resistance to frost action. The estimated porosities of these fragments were generally the same as those of the nonsusceptible fragments.

Thus, the analysis by SEM was useful in that it identified, qualitatively at least, pore-size range as an important factor in the resistance of aggregates to D-Cracking.

Scanning electron microscope analysis was also carried out as part of the Image Analysis study. SEM photos of the aggregates used in that study are contained in the publication by Garychuk (1986).

2.5.2 Transport Canada (Winnipeg) study

2.5.2.1 Introduction

Transport Canada recovered nine 150 mm diameter cores from the Winnipeg International Airport apron, which had undergone D-Cracking, and commissioned an independent laboratory to carry out a petrographical study of the core aggregates. The studies included: visual classification of aggregate type; microscopic petrography; x-ray diffraction; and scanning electron microscopy. The results, which were reported by Mysyk (1987) are presented in the following sub-sections.

2.5.2.2 Aggregate Classification

The aggregate for the concrete came from the Birds Hill pit and the maximum top size was 40 mm. The classification of the core aggregate was as follows:

6 cores - 80% carbonate, 15% granite, 5% volcanic

2 cores - 75% carbonate, 20% granite, 5% volcanic

1 core - 85% carbonate, 10% granite, 5% volcanic. This core was the most severely deteriorated.

2.5.2.3 Fracture study

Seventeen horizontal slabs were cut from the nine cores and were examined with a 40X stereo binocular microscope. All major fracturing was associated with carbonate aggregate retained on the No. 40 sieve.

2.5.2.4 Microscopic Petrography

Thin sections were made from all cores and were examined under a polarizing microscope with 400X magnification. Twenty-nine nondurable coarse carbonate particles were examined. Twenty of these particles consisted of homogeneous calcite or dolomite, while nine were heterogeneous in grain size consisting of varying amounts of carbonate minerals.

An examination of some durable particles indicated approximately the same grain size range as found in the nondurable particles. The crystalline structure of the durable particles was generally a little coarser than that of the nondurable particles.

2.5.2.5 X-Ray Diffraction

Ten nondurable fractured aggregate particles were recovered from the cores and were analyzed by x-ray diffraction. The results indicated that of the ten particles, 5 were dolomite, 4 were limestone, and 1 was a silicified limestone. This suggests that, for the particular carbonates examined, there was no apparent difference in the durability of calcite vis a vis dolomite, which points out that mineralogical composition alone is not necessarily the only factor that affects the resistance of aggregates to frost action.

2.5.2.6 Scanning Electron Microscope

Small fragments (less than 5 mm in diameter) were taken from the particles used in the x-ray diffraction study and were examined using a Cambridge Stereoscan Mark IIA scanning electron microscope. All fragments displayed common texture and porosity types. The limestones and dolomites were ". . . very finely, finely or rarely medium crystalline, equigranular mudstone, wackestone, or packstone". There were little or no clay minerals or sulfates nor was there any evidence of layering. The major type of porosity was "intergranular". The pore size range was ascertained to be narrow, a characteristic of aggregates susceptible to D-Cracking.

Under high magnification the silicified limestone indicated a heterogeneous texture, and the major porosity type was "intergranular".

The study supported the notion that carbonate aggregates with a narrow pore-size range are highly susceptible to D-Cracking.

2.6 Resistance of Aggregates to Frost Action

2.6.1 Introduction

The ASTM C-666 B test is the current standard test for determining the susceptibility of concrete to D-Cracking. The test, which is performed on concrete prisms, requires very close control of temperatures, and is very time consuming. Since D-Cracking originates with the coarse aggregate, it would be preferable to devise a test involving the aggregate only. As mentioned in Sections 1.4 and 1.5, some attempts have been made to test the aggregates' susceptibility to weathering, but as yet no universally acceptable standard test has been developed.

At the University of Manitoba, a preliminary study was carried out to determine if aggregate, known to be susceptible to D-Cracking, would fracture when exposed to freeze-thaw cycles, and thus provide a basis for a simpler method of test.

2.6.2 Aggregate Studied

Coarse aggregate from the Birds Hill pit, which had previously been determined to be susceptible to D-Cracking, was used in this investigation. The aggregate was obtained from a concrete supplier, and it had been processed to meet the required concrete specifications. It was a blend of carbonate rock (approximately 60% by weight) and igneous rock (40% by weight). The aggregate particle sizes ranged from 100% passing the 40 mm sieve to 100% retained on the 26 mm sieve.

2.6.3 Test Apparatus

A high pressure cell for the aggregate was fabricated from a mild steel cylinder. The cell had a diameter of 150 mm and a length of 300 mm. A steel plate, 20 mm thick, was welded to one end of the cylinder, forming the base. The top of the cell consisted of another 20 mm thick plate, held in place by 5 rods extending up from the base plate. An O-ring provided the seal between the top plate and the cylinder. The cell was equipped with a valve at the bottom and a valve-port combination at the top to facilitate evacuation and saturation of the aggregate. A thermocouple was mounted inside the cell, at heights of 25, 100, and 125 mm above the base, to monitor the temperatures, so as to know when the aggregates were in a frozen state. Control valves and pressure gauges were connected to the cell for monitoring and controlling cell pressures. A photo of two such cells is shown in Figure 2.21.

2.6.4 Test Procedure

Approximately 4500 to 5000 g of aggregate were placed in the cell and the top was securely fastened. The bottom valve was closed and the top port was connected to a vacuum pump. The sample was evacuated for 24 h under a negative pressure of 75 kPa. The bottom valve was then connected to a de-aired distilled water source. With the cell still under a vacuum of 75 kPa, water was allowed to flow into the cell at a rate of 2 mL per minute. It took about 8 hours to completely fill the cell.

At the end of the saturation process, the water supply

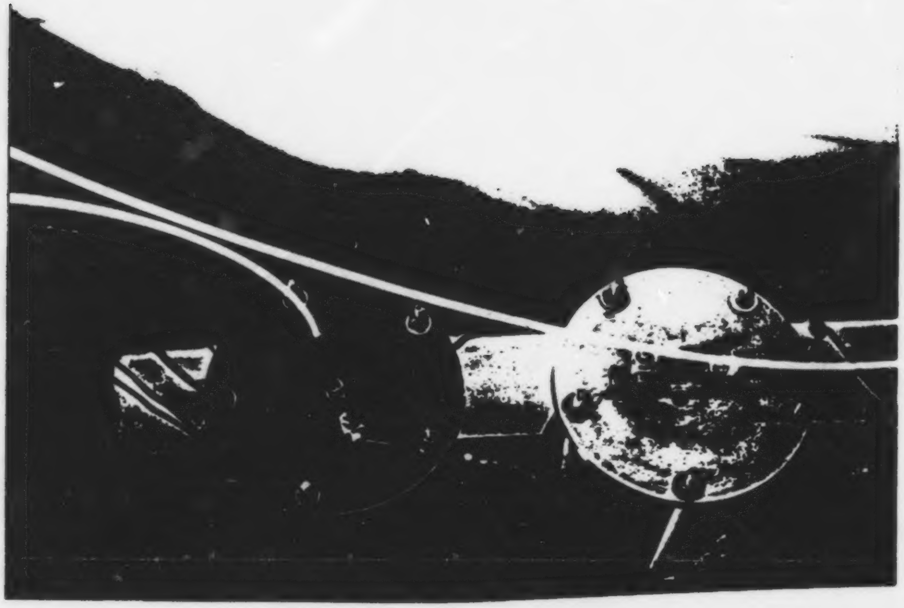


Figure 2.21 High pressure freeze-thaw cells

valve was closed and the vacuum was released very slowly. The top of the cell was then removed and the water level was lowered to approximately 6 mm above the surface of the aggregate sample.

The top of the cell was put back into place and positive cell pressures were applied. In one test the cell pressure was 1000 kPa, in a second test the cell pressure was 500 kPa and in a third test the cell pressure was left at atmospheric pressure.

For comparative purposes, a fourth test was set up in which there was no vacuum saturation and the aggregate was merely submerged in water. The cell pressure was left at atmospheric pressure as well.

The cells were then placed in a freezing cabinet and the thermostat was set at -20°C. Thermocouple readings were taken periodically and it was found that it took approximately 12 hours to freeze the samples. It also took about 12 hours to thaw the samples and therefore a freeze-thaw cycle was completed every 24 hours.

After 20 freeze-thaw cycles the water was drained from the cells and the cells were placed in an oven (105°C) to dry. A sieve analysis was then carried out on each sample. The samples were then replaced in the cells, the saturation procedure was repeated and the samples were subjected to additional freeze-thaw cycles. The samples were dried and sieved at 20, 30, 40, and 50 freeze-thaw cycles.

2.6.5 Test Results

1. It was observed that, even after 40 freeze-thaw

cycles, none of the igneous aggregate particles had fractured.

2. Some of the carbonate aggregate particles fractured as the result of the freeze-thaw environment. In order to obtain a meaningful assessment of the carbonates' susceptibility to fracturing, all calculations of aggregate breakdown were based on the carbonate content of the samples. Furthermore, the only quantitative means of assessing the breakdown of aggregate, was to calculate the percent of aggregate particles passing the 26 mm sieve at the end of the test. The results, given in Table 2.10, indicate that the particular carbonates tested, underwent substantial breakdown as the result of repeated freeze-thaw cycles. However, there was no apparent correlation between the percentage of breakdown and the conditions of saturation, other than, generally speaking, evacuation led to more particle breakdown than did simple soaking of the aggregate. No conclusion could be drawn regarding the role of positive saturation pressures in aggregate fracture.

Table 2.10: Breakdown of Aggregates Due to Freeze-Thaw

TEST CONDITIONS	PERCENT PASSING 26 mm SIEVE			
	20 CYCLES	30 CYCLES	40 CYCLES	50 CYCLES
Evac. & 1000 kPa	8	13	16	21
Evac. & 500 kPa	21	26	31	33
Evac. & 0 kPa	9	15	22	26
Soak. & 0 kPa	6	13	13	-

The grain-size distribution curves for each sample at the start of the test and after 50 cycles are shown in Figure 2.22. The shaded areas serve as an indiction of the extent of the breakdown of the aggregates. The larger the area, the more extensive the breakdown. In this respect, the size of the shaded area of the sample that was not evacuated was noticeably smaller than the others. There was no apparent correlation between size of shaded area and magnitude of positive saturation pressures. Since not all aggregates were susceptible to fracture under freeze-thaw cycles, a much larger sample would be required, or many more tests would have to be conducted, in order to properly examine the possibility of there being such a correlation.

Photographs of one of the samples after 50 freeze-thaw cycles are shown in Figure 2.23. The sample was separated into aggregates of igneous and carbonate origin. As can be seen (Figure 2.23a), there was no fracturing of the igneous aggregates and substantial fracturing of the carbonate aggregates. A close-up of the fractured aggregates (Figure 2.23b) shows evidence of a complete breakdown of some aggregates rather than just a fracturing of the large particles. This has important implications in that it appears to dispute the concept that D-Cracking can be eliminated by simply reducing the top size. Obviously, the nondurable aggregates continued to break down even as the particle size became smaller and smaller.

2.6.6 Conclusions

The idea of testing the resistance of aggregates to frost action, by means of freeze-thaw tests conducted on the

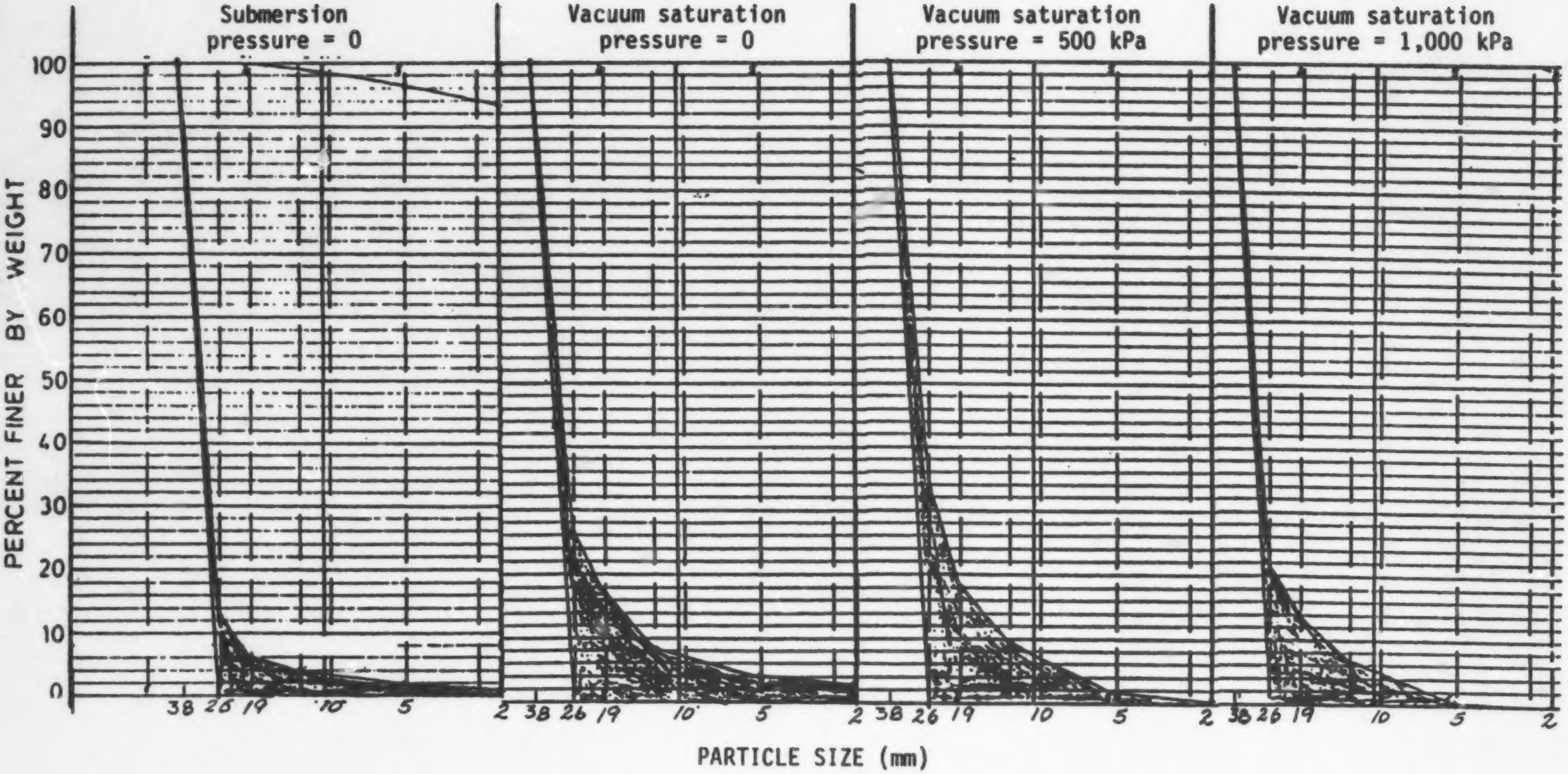
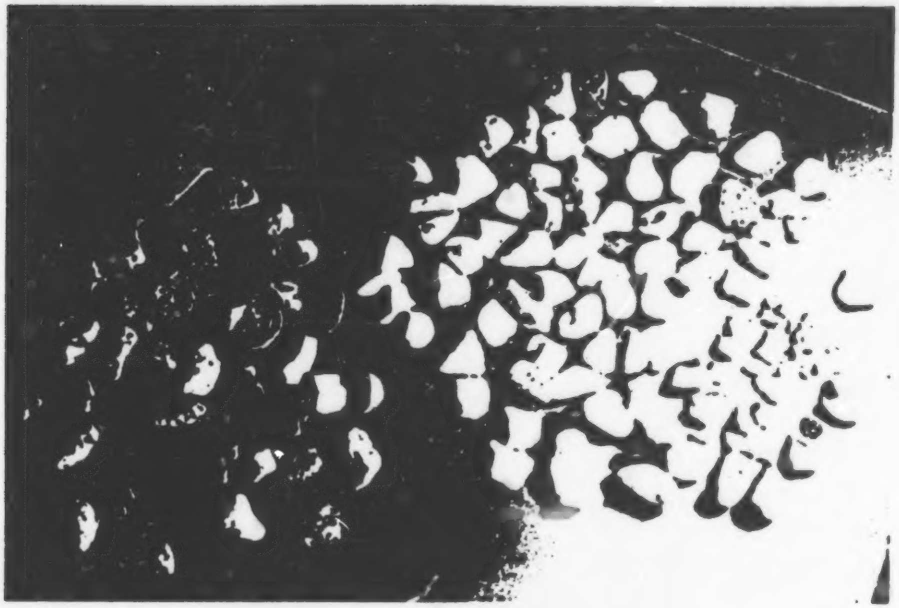
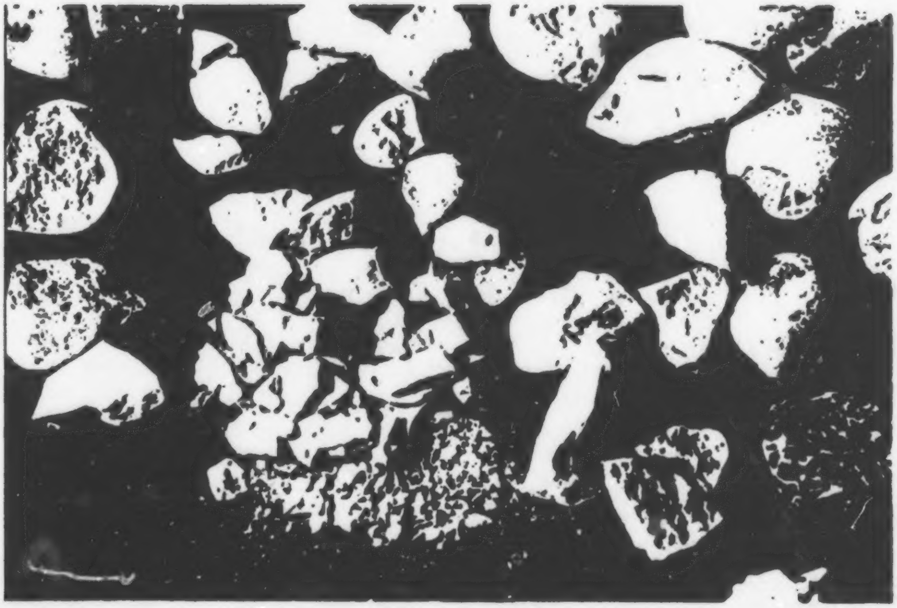


Figure 2.22 Grain-size distribution before and after 50 freeze-thaw cycles



(a)



(b)

Figure 2.23 Photographs of samples after 50 freeze-thaw cycles: (a) entire sample of igneous and carbonate particles, (b) close-up of fractured carbonate particles

aggregates alone, appears to have merit.

Vacuum saturation of the aggregates appeared to increase the susceptibility of the aggregate to fracture, suggesting that it is an effective means of getting water into the fine pores.

The results of the tests were unable to show any benefit in using a positive saturation pressure in combination with evacuation, in assessing the aggregates' susceptibility to frost action.

Fracturing or breakdown of aggregate particles was not limited to large particles. Fracturing appeared to continue, reducing the coarse aggregate particles to sand size.

CHAPTER 3

MANITOBA SURVEYS

3.0 INTRODUCTION

Our survey has covered four of the six districts in the City of Winnipeg. District 1 has generally been covered with asphalt or the roads have been replaced in the last few years. District 4 in the Transcona area has not been covered, being the smallest of the districts. A cursory review showed conditions to be of a similar nature to those in the other districts on which we have reported. Therefore, Districts 5 and 6, 2 and 3, as covered, represent the majority of Winnipeg streets where the concrete has been in service for 15 to 30 years. In addition, the concrete roads, not asphalt covered, and the major airports have been surveyed.

Our examination has been visual and the judgments as to the state of the concrete, while subjective, are related to a "standard", as shown in the following Fig. 3-1. The ratings being used are on a scale of 0 to 3 where 3 represents major damage.

Fig. 3-2 represents the D-Cracking Survey Chart used. Pictures representing the more serious damage at the cross-points under consideration were taken. Fig. 3-3 represents the survey nomenclature.

3.1 Purpose:

The purpose of our state-of-the-art survey is to report on the extent and seriousness of D-Cracking of concrete in Manitoba, to suggest possible remedial measures, to explore the need for research, and to supply a complete bibliography.

The magnitude of the D-Cracking problem in Manitoba is covered in this chapter, in the following order:

3.2 Observations in the City of Winnipeg:
  (1) Districts Nos. 5 and 6
  (2) Districts Nos. 2 and 3

3.3 Graphical Analysis of each District Survey:
  (1) Occurrence of D-Cracking: Frequency vs Age
  (2) Severity of D-Cracking: Distress vs Age
  (3) Frequency of Occurrence vs Severity
  (4) Composite of Frequency of Occurrence vs Severity covering the 4 districts surveyed.

3.4 Observations on Concrete Highways in Manitoba (not paved over).

3.5 Observations on airports in Manitoba:
  (1) CFB, Winnipeg
  (2) CFB, Portage
  (3) Rivers Airport
  (4) Churchill Airport

3.6 Photographic Descriptions

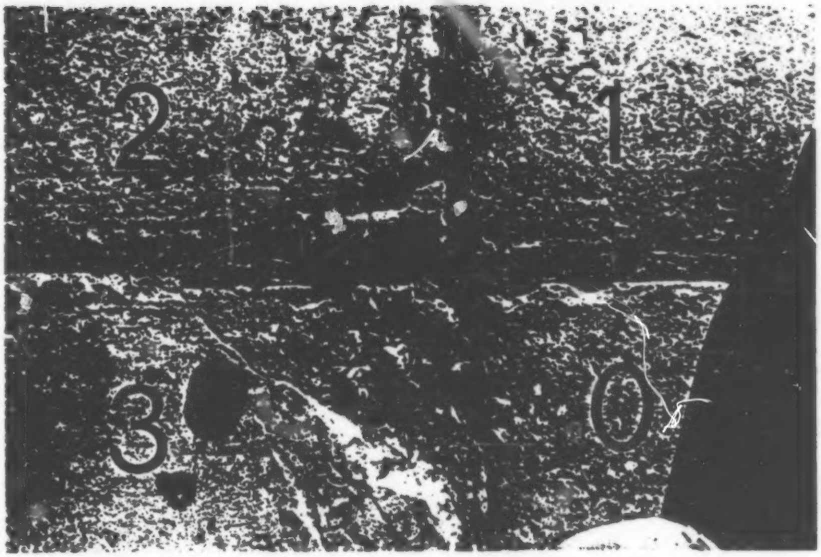


FIG. 3-1

"Standard" used in this Report
Originated by G.J. Hermanson
(M.Sc. Thesis, University of Manitoba, 1987)

D-CRACKING SURVEY

CITY OF WINNIPEG

DISTRICT NO. ____

DATE ______________

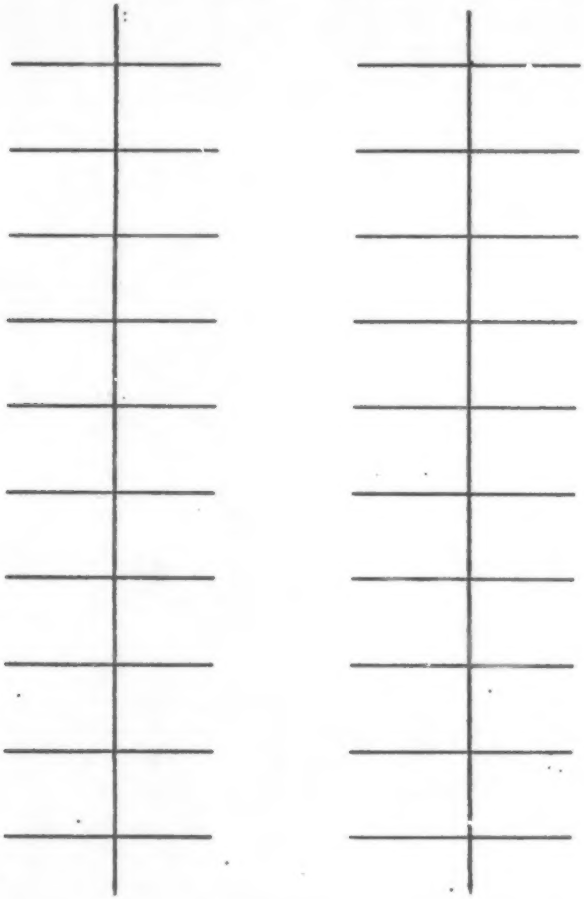
NAME ______________

STREET ____________________

YR. OF CONSTR. _____

From ____________________ to ____________________

Percent joints affected ____ Average distress severity ______



COMMENTS ____

____

____

____

FIG. 3-2

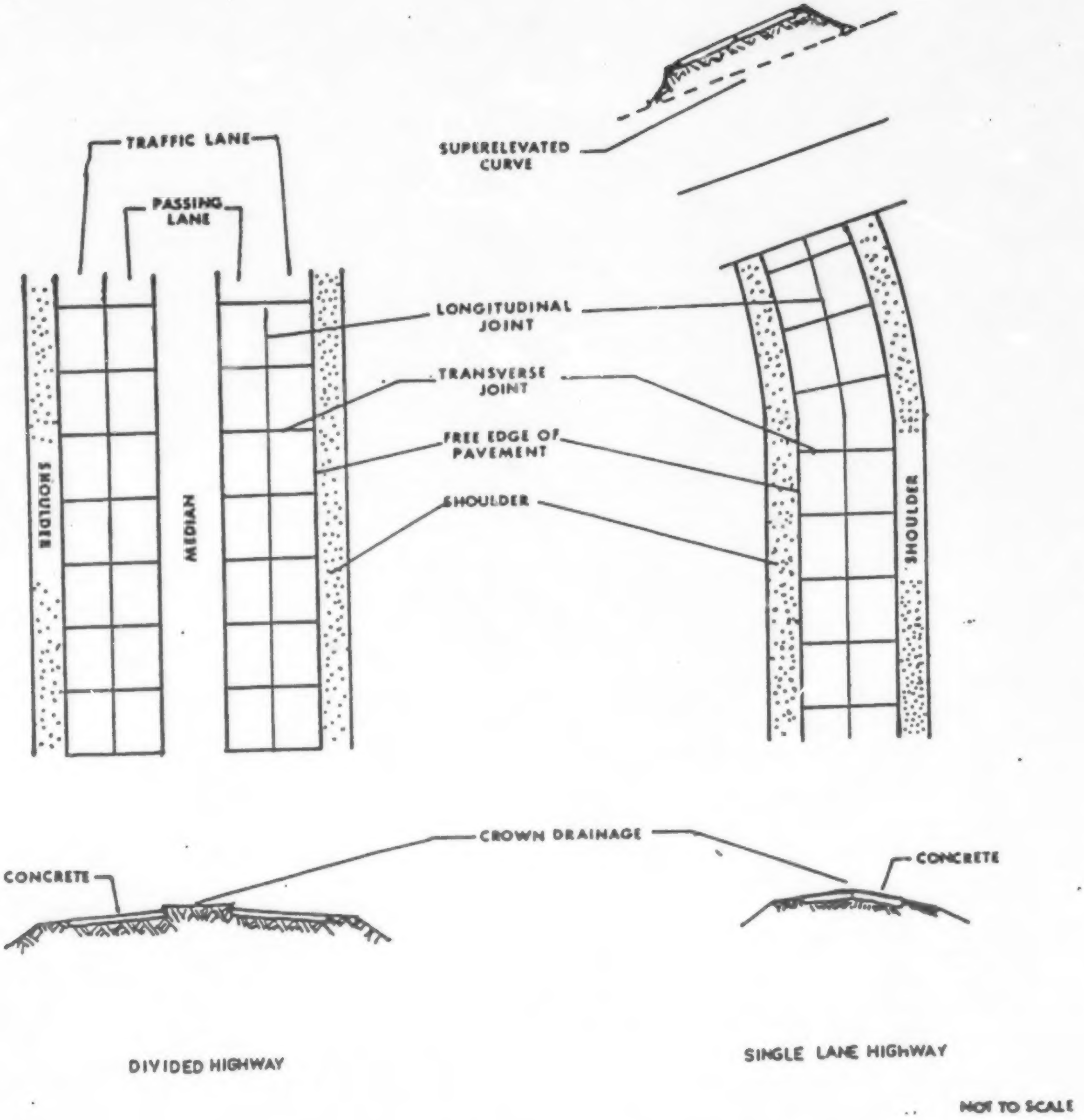

TRAFFIC LANE
PASSING LANE
SUPERELEVATED CURVE
LONGITUDINAL JOINT
TRANSVERSE JOINT
FREE EDGE OF PAVEMENT
SHOULDER
SHOULDER
MEDIAN
SHOULDER
CROWN DRAINAGE
CONCRETE
CONCRETE
DIVIDED HIGHWAY
SINGLE LANE HIGHWAY
NOT TO SCALE

Figure 3-3: Survey nomenclature

3.2 Observations, City of Winnipeg:

3.2.1 Districts Nos. 5 and 6 -- Observations:

District 5 is substantially east of the Red River, covering the communities of St. Boniface and St. Vital. District 6 is north of the Assiniboine River and west of the Red River, covering the communities of Assiniboine Park and Fort Garry. (See Map of Winnipeg, Fig. M-1, Area 5 and 6).

The pavements were generally in poor to very poor condition. D-Cracking was seldom the predominant form of concrete deterioration. Other prominent forms of deterioration included:

Scaling (deterioration of the surface finishing);

Popouts (surface voids caused by aggregates "popping" out of the concrete);

Spalling (break-away of concrete at joints and cracks, not associated with D-Cracking);

Structural cracks (longitudinal, transverse, and diagonal). Perhaps the most common and most severe form of deterioration. Often the cracks had an opening of 25 to 50 mm suggesting very large tensile forces transmitted by the supporting soil. Faulting was also common which contributed to spalling, etc.;

Ravelling (deterioration of the mortar, thereby the coarse aggregate breaking loose);

Spalling of concrete where reinforcement was placed at too shallow a depth.

Deterioration at manholes (including D-Cracking) was generally more severe than at the joints.

D-Cracking occurred along structural cracks in some instances, primarily where the structural crack intersected a transverse joint.

D-Cracking was often limited to two adjoining slabs on one side of the longitudinal joint only.

In some cases D-Cracking was limited to one slab only.

The severity of D-Cracking of the four slabs at a joint intersection often varied considerably.

There was a lot of evidence of vertical movements of slabs or portions thereof. In some cases the slabs had obviously heaved, whereas in other cases, they had obviously settled.

3.2.2 Districts Nos. 2 and 3 -- Observations:

Districts 2 and 3 are respectively west and north of the downtown area of Winnipeg, bordered on the south by the Assiniboine River and on the east by the Red River, covering respectively St. James-Assiniboia and Lord Selkirk-West Kildonan. (See map of Winnipeg, Fig. M-1, Areas 2 and 3).

The pavements in the two districts are in poor condition. Our surveys were taken on the condition of the concrete at the intersection of four slabs, on the centre-line of the road. Often this location would read zero defect, although longitudinal cracks would be evident, paralleling the centre-line halfway down the slab (Fig.D3-10).

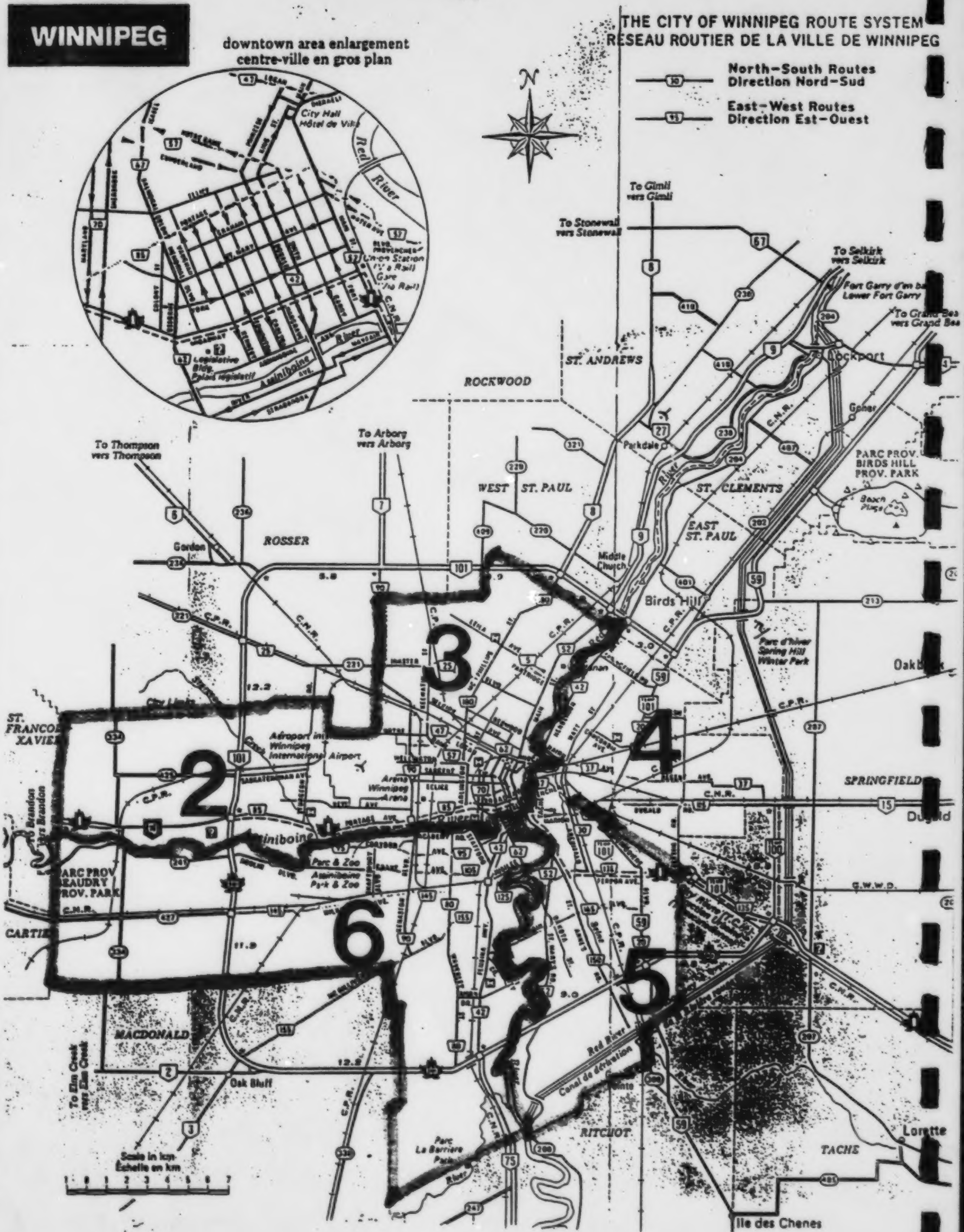

WINNIPEG
downtown area enlargement
centre-ville en gros plan
THE CITY OF WINNIPEG ROUTE SYSTEM
RESEAU ROUTIER DE LA VILLE DE WINNIPEG
North-South Routes
Direction Nord-Sud
East-West Routes
Direction Est-Ouest
N
City Hall
Hôtel de Ville
Red River
Union Station
Legislative Bldg.
Palais législatif
Assiniboine River
To Gimli
vers Gimli
To Stonewall
vers Stonewall
To Selkirk
vers Selkirk
Lockport
ST. ANDREWS
ROCKWOOD
To Thompson
vers Thompson
To Arborg
vers Arborg
Parkdale
WEST ST. PAUL
ST. CLEMENTS
EAST ST. PAUL
PARC PROV. BIRDS HILL PROV. PARK
ROSSER
Gordon
Middle Church
Birds Hill
Oakbank
Parc d'hiver Spring Hill Winter Park
3
2
4
5
6
City Limits
ST. FRANCOIS XAVIER
Aéroport int. Winnipeg International Airport
Arena Winnipeg Arena
SPRINGFIELD
Dugald
To Brandon
vers Brandon
PARC PROV. BEAUDRY PROV. PARK
Parc & Zoo Assiniboine Park & Zoo
CARTIER
MACDONALD
Oak Bluff
To Elm Creek
vers Elm Creek
Scale in km
Échelle en km
Parc La Barrière Park
RITCHOT
Red River
Canal de dérivation
TACHE
Lorette
Ile des Chenes

FIG. M-1

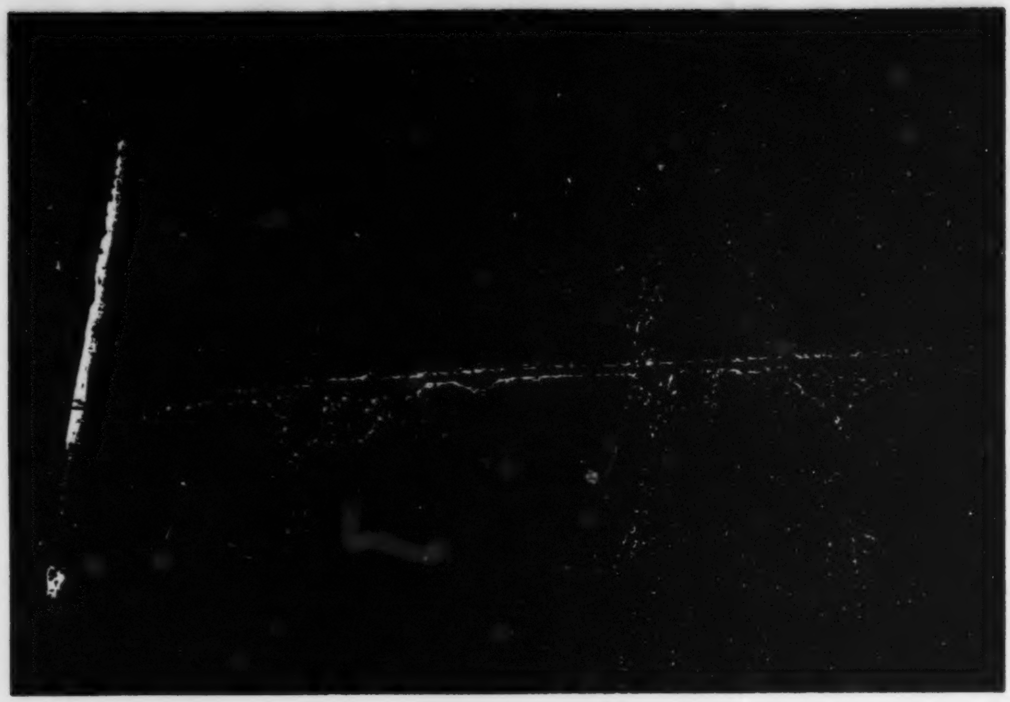


FIG. D3-10

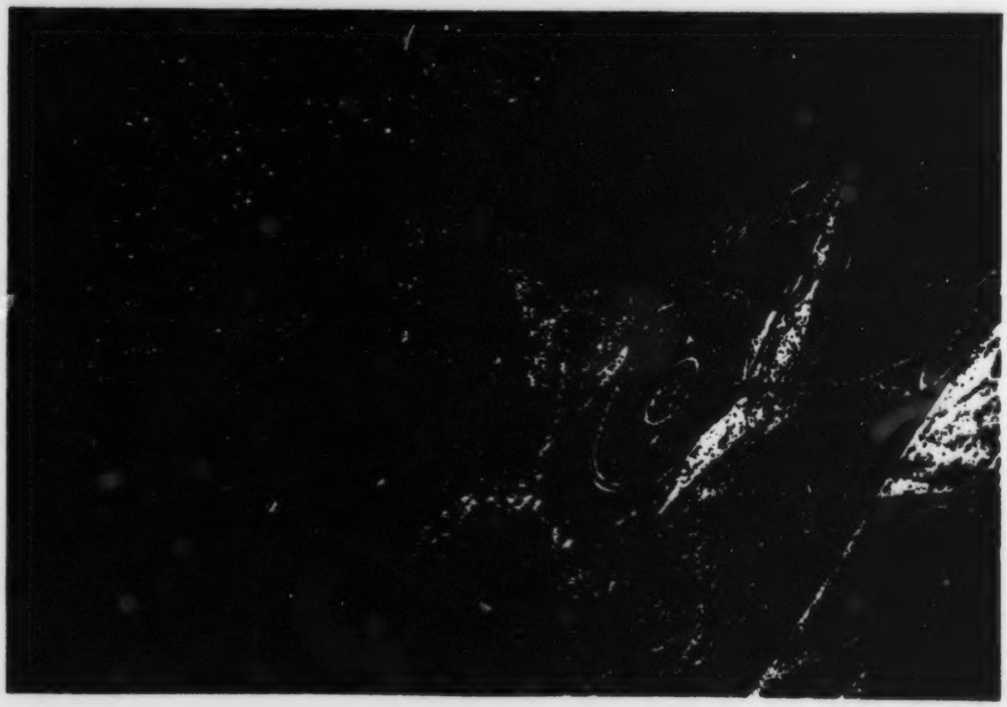


FIG. D2-7

D-Cracking, while evident, was not the only form of damage. D-Cracking occurred along structural cracks, particularly at the intersection of transverse joints, showing up in one, two, three, or all four joining edges on occasions.

Other forms of damage included:

(i) Severe cracking around manhole covers (Figs. D2-7, D3-2);

(ii) Spalling where the surface of the concrete has come away, exposing the aggregate (Fig. D2-8);

(iii) Scaling where the steel reinforcing bars have been too close to the surface and the resulting contraction/expansion and rusting of the bar has caused removal of the concrete surface (Fig. D3-5);

(iv) Structural cracks (in all directions) which appear to be caused by severe tensile forces in the underlying soil as a result of our normal extremes of weather. (Fig. D3-6);

(v) The heaving and settling of concrete slabs expose the edges to heavier traffic forces and increased breakdown.

Portage Avenue West in Winnipeg:

This road has one of the heaviest traffic loads in the city. It is one of the first roads cleared and sanded/salted in the winter. The intersections and most of the avenue have

FIG. D3-2

FIG. D2-8

FIG. D3-5

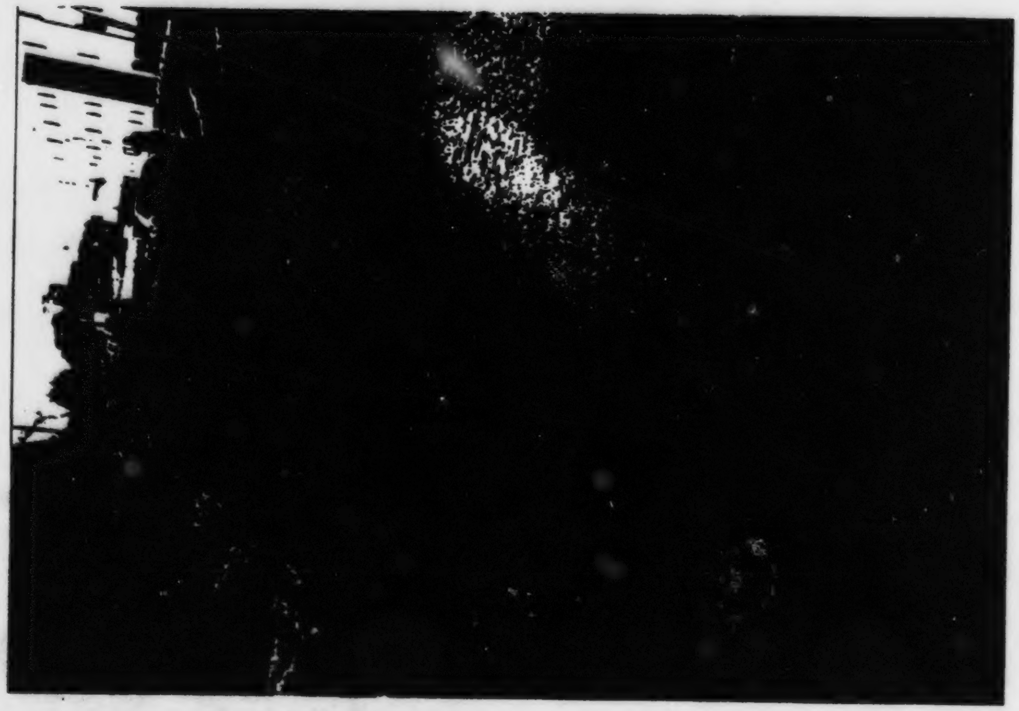


FIG. D3-6

been asphalted over. The following three pictures show consecutive manhole covers on Portage Avenue going west, just east of Mount Royal Street (Figs. D2-10, D2-11, D2-12).

FIG. D2-10

FIG. D2-11

FIG. D2-12

3.3 Tabulated and Graphical Analysis of D-Cracking of Concrete by Districts in the City of Winnipeg and a Composite Graph.

See Appendix B.

3.4 Observations on D-Cracking of Highways in Manitoba (not paved over):

3.4.1 Concrete Highways in Manitoba (not paved over), (Fig. M-2):

Hwy 2	Perimeter to Hwy 13 (Oak Bluff to Elm Creek)
Hwy 3	Carman to Perimeter
Hwy 7	Perimeter to north of Hwy 67
Hwy 8	North of Perimeter to 12.3 km north of PH 67
Hwy 14	Hwy 75 to Hwy 30
Hwy 15	Hwy 207 to west of Anola. This highway was surveyed by Hermanson 1986.
Hwy 23	From Hwy 75 to Hwy 3
Hwy 30	Altona, from Poplar Drive to Hwy 14
Hwy 59S	From Perimeter to 1 km past PH 405
Hwy 59N	2 km north of Birdshill Park to 1.1 km north of PH 44
Hwy 75S	From Perimeter to just south of St. Norbert

3.4.2 Summary of Highway Survey:

Hwy No.	Const Yr.	Mile (km)	No. Surv.	Occurrence (%) Range	Occurrence (%) Av.	Severity Range	Severity Av.
1W	1968-74	12	5	35-80	58	0.69-1.53	1.13
2	1971-74	45	8	0-10	4	0-0.1	0.04
3	1962-64	52	11	15-78	44	0.22-1.65	0.82
7	1980-82	18	6	0-6	2	0-0.1	0.04
8N	1971-73	13	5	0-3	6	0-0.1	0.08
59NN	1967	7	4	65-80	70	1.4-1.6	1.50
59NS	1967	7	6	40-50	45	0.40-0.85	0.76
14	1957	14	7	91-100	97	2.16-3.00	2.54
23	1966-68	24	8	8-91	28	0.15-2.54	0.60
59S	1959R 1972L	12	5	40-78	60	0.69-1.75	1.20
75S	1967-78 1984-85	5	5	34-85	66	0.79-2.45	1.43

HIGHWAY 2 (1971, 1972, 1973, 1974):

Survey from just west of Oakbluff to Elm Creek.

There were quite a few structural cracks running parallel and close to the centre-line joint. The concrete tended to spall between the crack and the joint, which made it difficult to distinguish between spalling and D-Cracking. As well, the breaking away of concrete at some joints appeared to be associated with small diagonal cracks across the joint rather than cracks parallel to the joint. The general impression was that the failures at the joints were not D-Cracking failures.

HIGHWAY 75S (1967, 1978, surfaced 1987):

Only a short section, from the Perimeter to just south of St. Norbert was available for inspection. The balance had an asphalt overlay.

There was considerable patching and replacing of joints. The roadway was generally in very poor condition. Where the joint had been replaced it was categorized as #3 severity.

HIGHWAY 59S (1959, 1972):

Survey started at Isle de Chien and proceeded northward to Perimeter.

The surface was covered with a thick seal coat which meant that only major deterioration could be readily observed. A #1 severity rating meant that only one or two minor cracks had reflected through the seal coat. Consequently, the overall ratings may have been less severe than the actual condition.

There was some D-Cracking of the transverse joint at the pavement edge, but generally this was less frequent and less severe than at the centre-line joints.

HIGHWAY 14 (16.7 km 1957; 27.2 km 1962; 14.2 km 1963;
21.2 km 1964):

Surface covered with a thin seal coat. Open cracks in the pavement reflected up through the seal coat. Fine cracks, as those associated with Category #1, would not be visible. Hence, the overall pavement rating could be on the low side.

The D-Cracking patterns were very well developed. The seal coat appeared to help hold the concrete in place in spite of the extensive cracking.

HIGHWAY 23 (29.1 km 1966; 0.6 km 1968):

Highway very recently paved from Hwy 75 to Hwy 326.

Survey started at Hwy 326.

Transverse joints cut at an angle, approximately 10 degrees from the perpendicular.

D-Cracking well developed in many instances along the transverse joint at the pavement edge.

Some major structural cracks in the longitudinal direction with well developed D-Cracking patterns at the intersection of these cracks and transverse joints.

Fairly extensive patching of the roadway surface with asphaltic concrete.

Generally, the roadway surface had a hard smooth finish.

HIGHWAY 3 (6.3 km 1963; 27.2 km 1962):

Survey started just out of Carman.

Fairly extensive patching and replacing of joints.

At some sections the D-Cracking was more severe at the pavement edges than along the centre-line.

Generally, it appeared that D-Cracking was more pronounced along superelevated curves than on the straight-of-way.

HIGHWAY 7N:

This highway is in very good condition, showing only 34 cracks in the pavement over a distance of 18 km. Most of the cracks were parallel to the longitudinal centre intersection as shown in photograph Fig. H7-1. A few cracks were across a pad away from a transverse joint. Three of the cracks had been asphalt filled.

Coming south on the west lane on a survey of 30 intersections, one longitudinal crack was found, Fig. H7-2.

The road on twin bridges on Highway 7 is deteriorating as shown in Fig. H7-3, H7-4 (7S) and Fig. H7-5, H7-6 (7N).

HIGHWAY 8N:

Highway 8 North of the Perimeter (Highway 101) for 12.7 km was in excellent condition (1978-79); then, for 0.4 km, the road was asphalt covered with many transverse cracks and breakout, without a centre line.

Going north from PH 67, Highway 8 for 12.3 km was again in excellent condition (1986-87). As on Highway 7, the only cracks were longitudinal as in Fig. H8-1 and these were rare.

HIGHWAY 1W:

Highway 1 West has generally been asphalt covered. For 55 km west of Winnipeg the concrete on the north road going west has widened at the centre-line and been filled with asphalt.

PORTAGE BYPASS:

At the time of the survey (August, 1988), this road was in poor condition for the most part and was being repaired on both the north and south roads. (See Map, Fig. M-3).

Figs. PBP-1 to 10 illustrate the general condition.

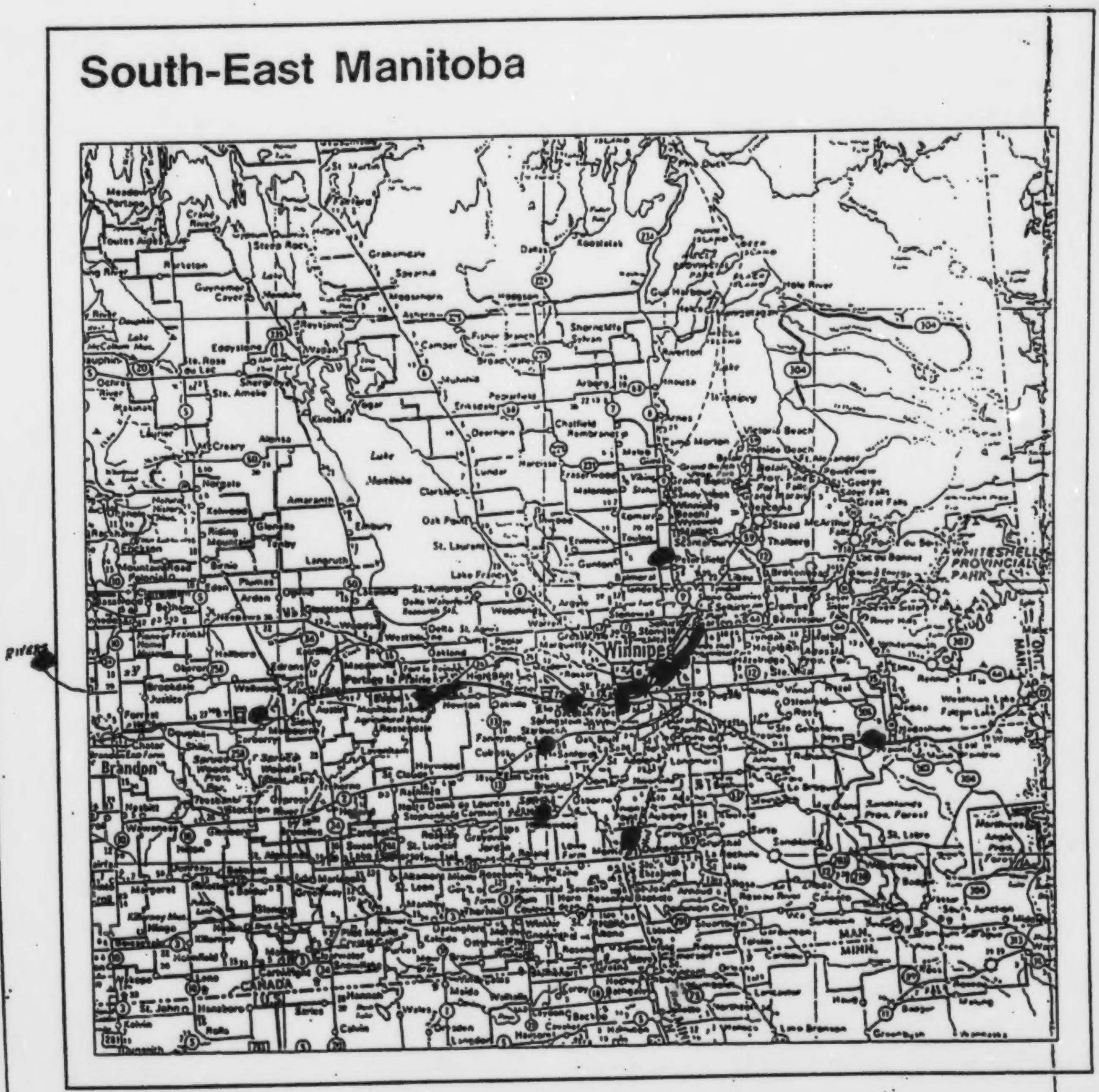
HIGHWAY 59:

Highway 59, where it runs north-east of the city, was installed in 1967 and has had a lot of repair work, particularly over the last few years. Even so, Highway 59 NN shows, from measurements taken at the intersection of longitudinal and transverse joints, to be in poor condition. A high percentage of the joints are damaged, as shown in Figs. H59-1, H59-5, and some edge damage is evident as shown in Fig. H59-2. The traffic lanes have had extensive repairs (Fig. H59-3). The road is somewhat rhythmically bumpy at speed due to lifting and lowering of the pavements.

Highway 59 NS is in slightly better condition than 59 NE over the same length of 7 km. Damage is again evident at the interior joints as shown in Fig. H59-4. Highway 59N, as it comes into Winnipeg, becomes Lagimodiere Boulevard. This boulevard is a heavy traffic area. It is deteriorating, but has had extensive replaced sections and areas covered with asphalt.

BRIDGES:

The bridge roads over the Red River Floodway are in good condition. The roads on either side of the bridge over the floodway on Highway #1 East are in poor condition with much



South-East Manitoba

Rivers
Winnipeg
Brandon
Lake Manitoba
Portage la Prairie
Whiteshell Provincial Park
Canada
Man.
Minn.

FIG. M-2

patching.

The Bishop Grandin Bridge and the St. James Bridge are in good condition, the latter having been recently repaired.

The Nairn Avenue Overpass, the Midtown Bridge, and the Disraeli Freeway surfaces are in good condition.

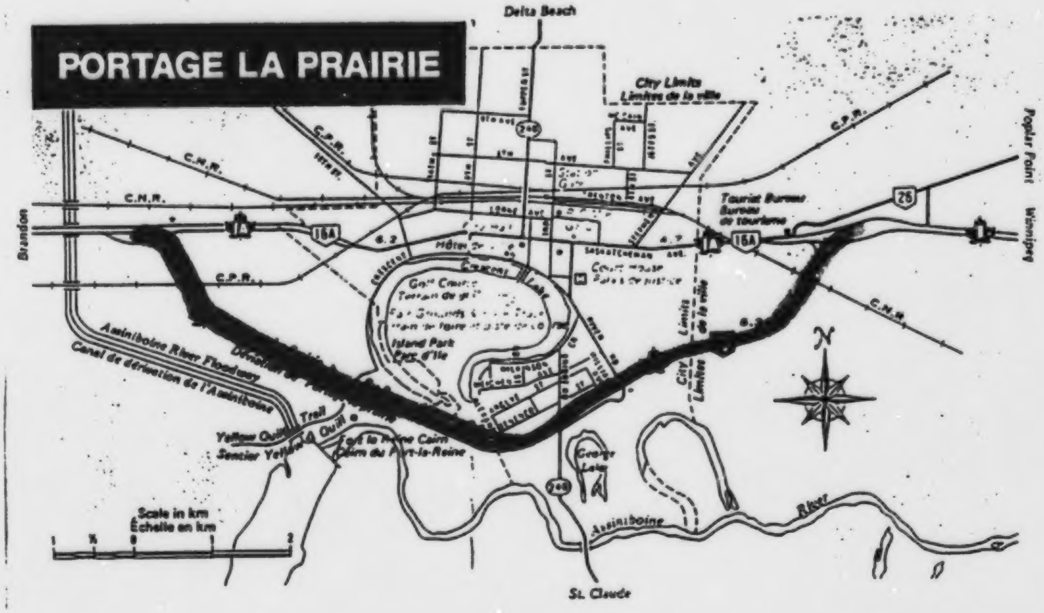

PORTAGE LA PRAIRIE
Delta Beach
City Limits
Limites de la ville
C.P.R.
C.N.R.
C.N.R.
C.P.R.
C.P.R.
C.N.R.
Poplar Point
Winnipeg
Brandon
1A
16A
1A
16A
26
1
240
240
Tourist Bureau
Bureau de tourisme
Golf Course
Island Park
Parc d'île
Crescent Lake
City Limits
Limites de la ville
Assiniboine River Floodway
Canal de dérivation de l'Assiniboine
Yellow Quill Trail
Sentier Yellow Quill
Fort la Reine Cairn
Cairn du Fort-la-Reine
George Lake
Assiniboine River
St. Claude
N
Scale in km
Échelle en km
1
½
0
1
2

FIG. M-3

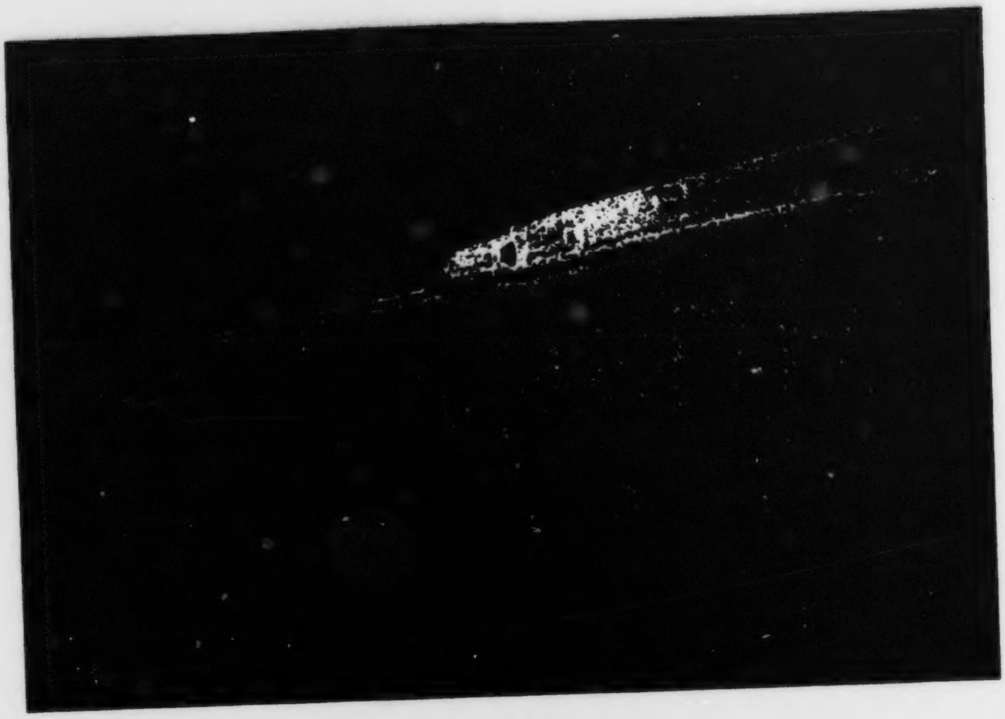


FIG. H7-1

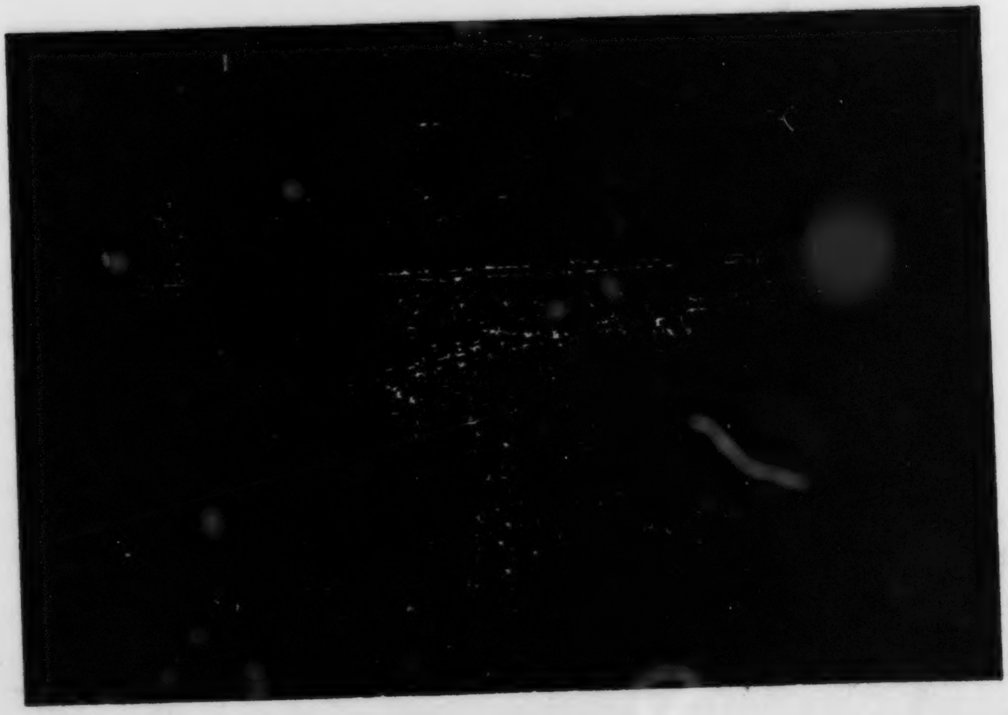


FIG. H7-2

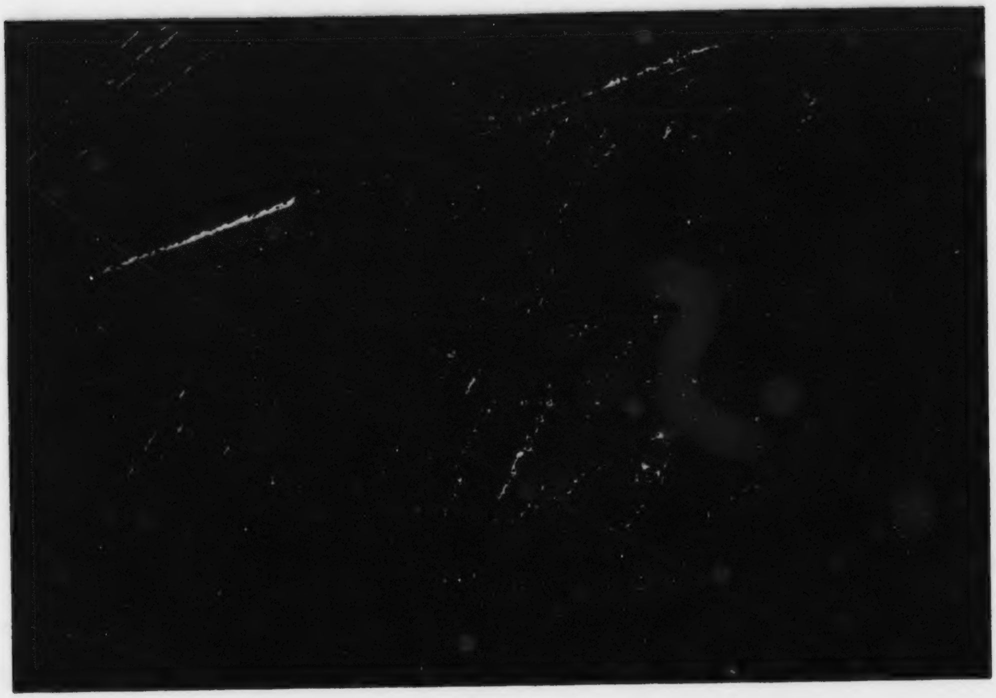


FIG. H7-3

FIG. H7-4

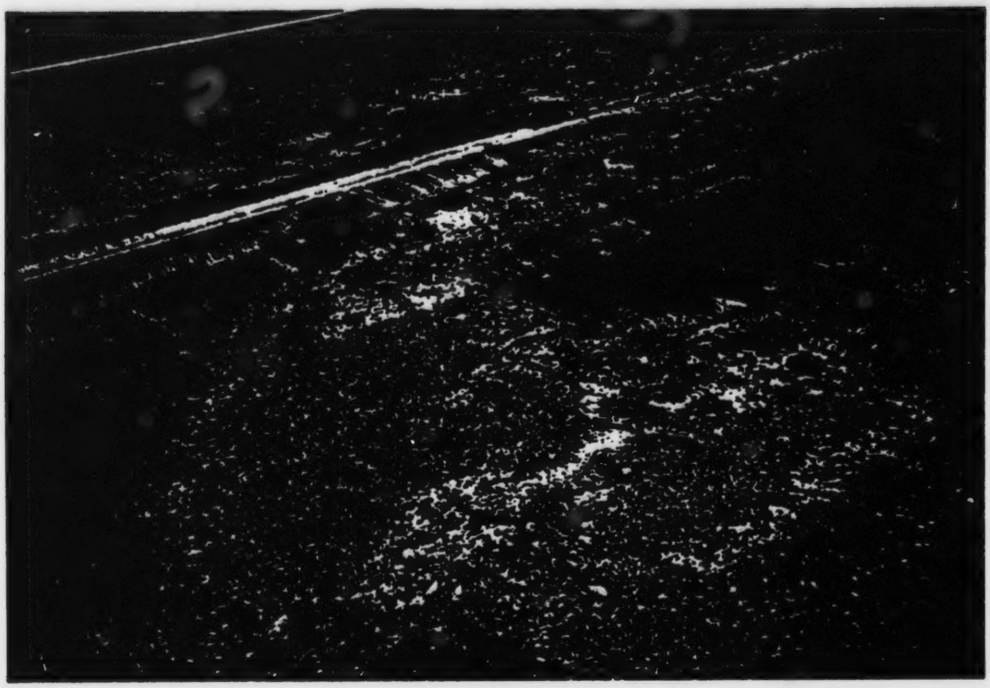


FIG. H7-5

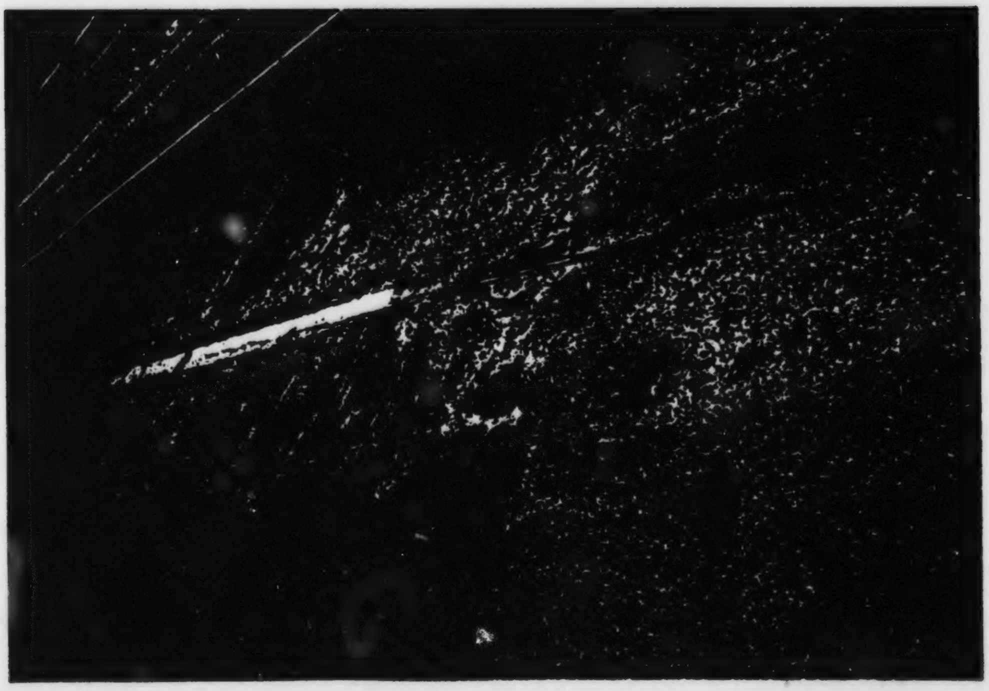


FIG. H7-6

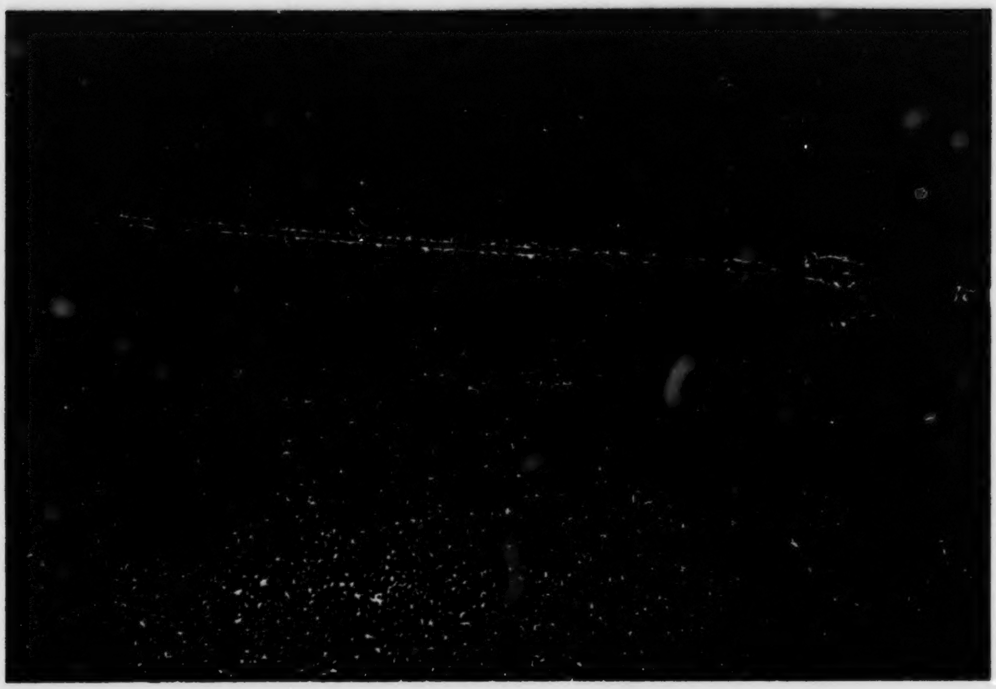


FIG. H8-1

FIG. PBP-1

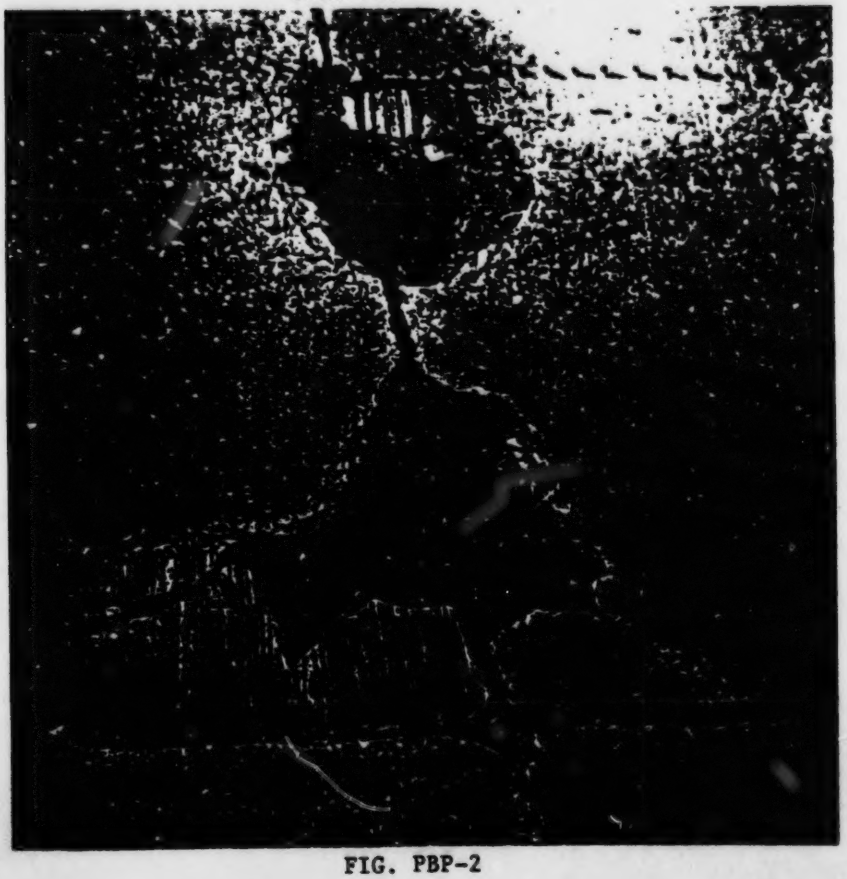


FIG. PBP-2

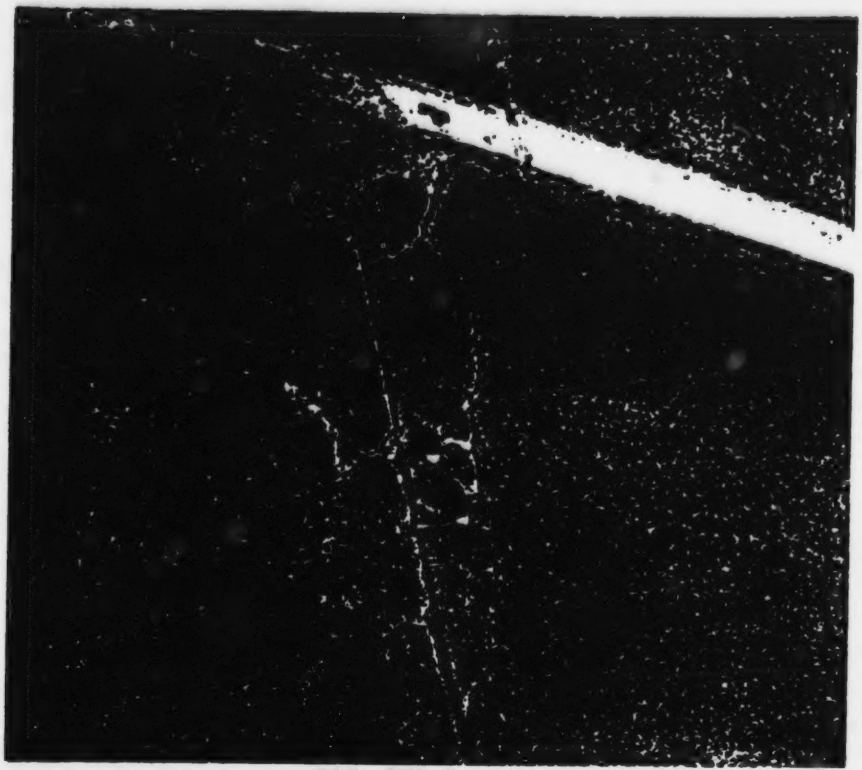


FIG. PBP-3

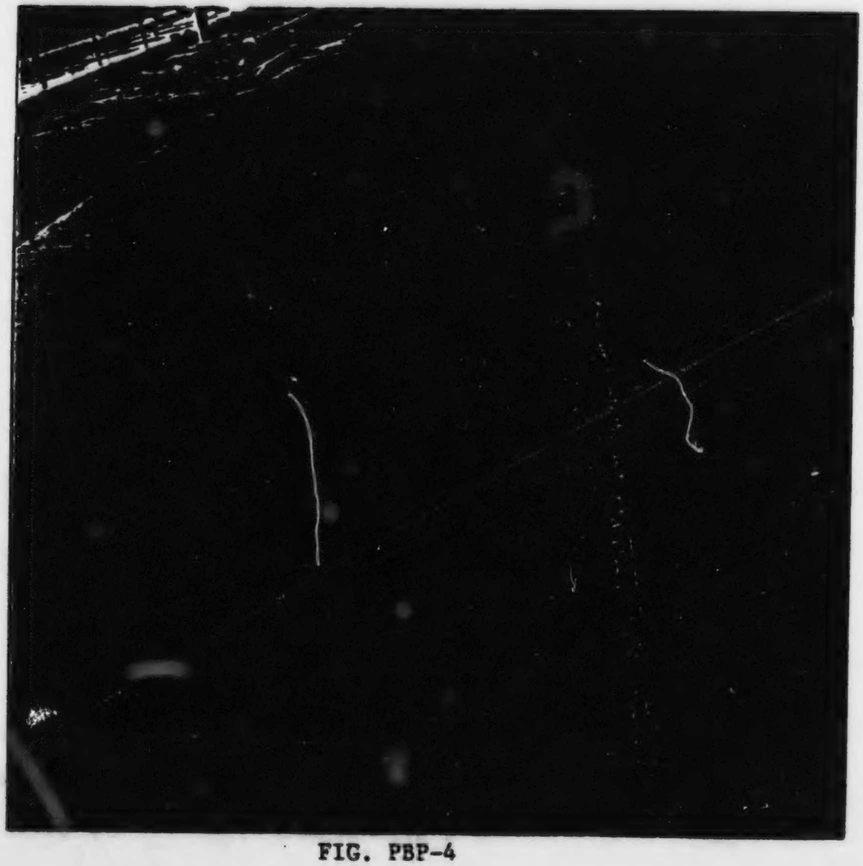


FIG. PBP-4

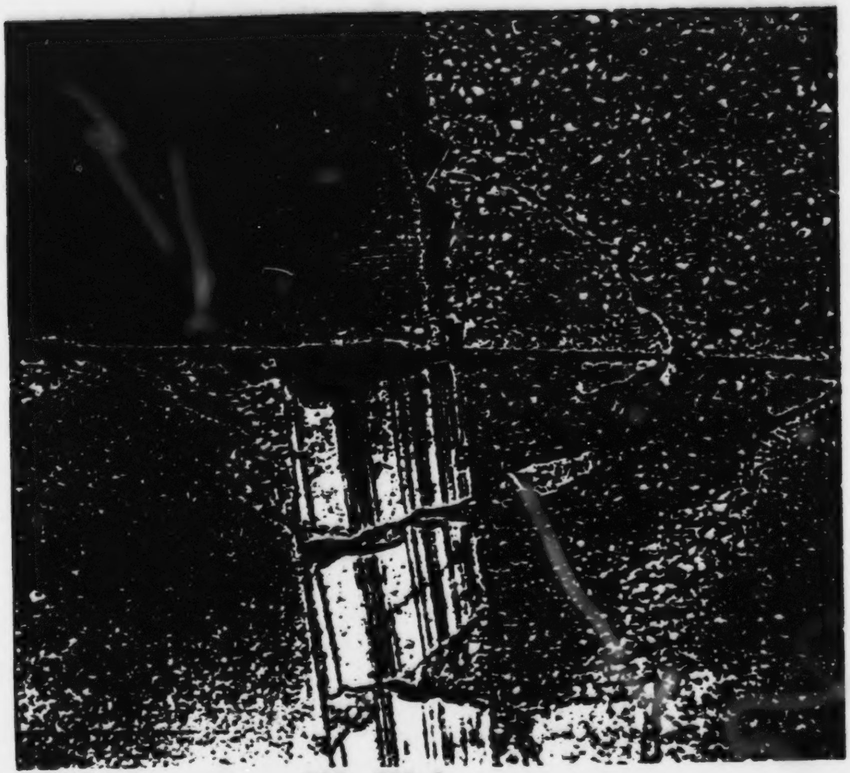


FIG. PBP-5

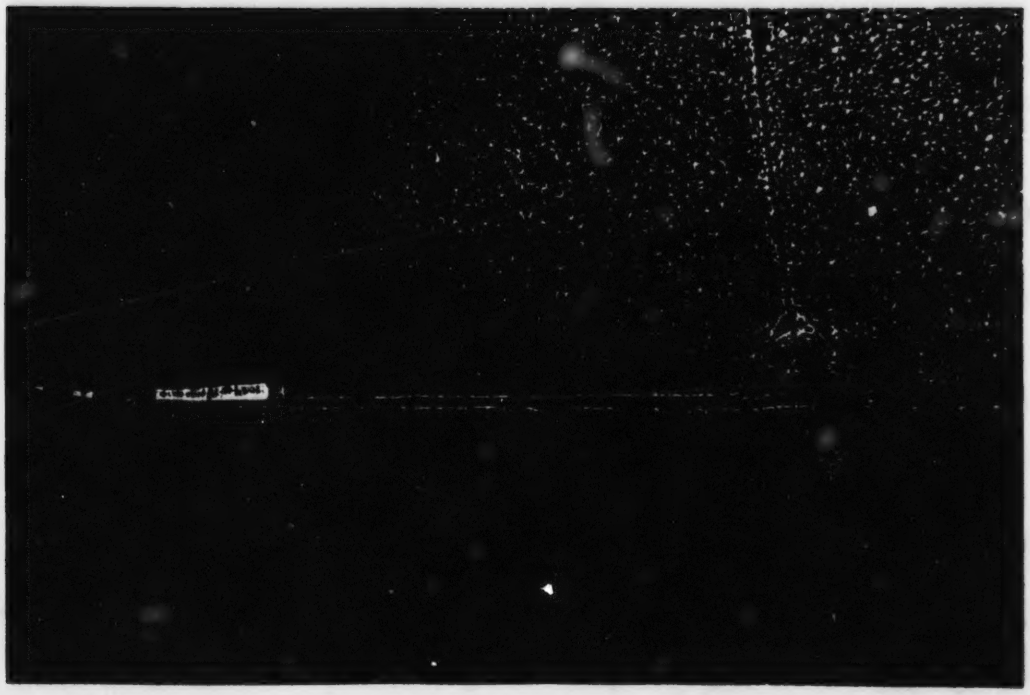


FIG. PBP-6

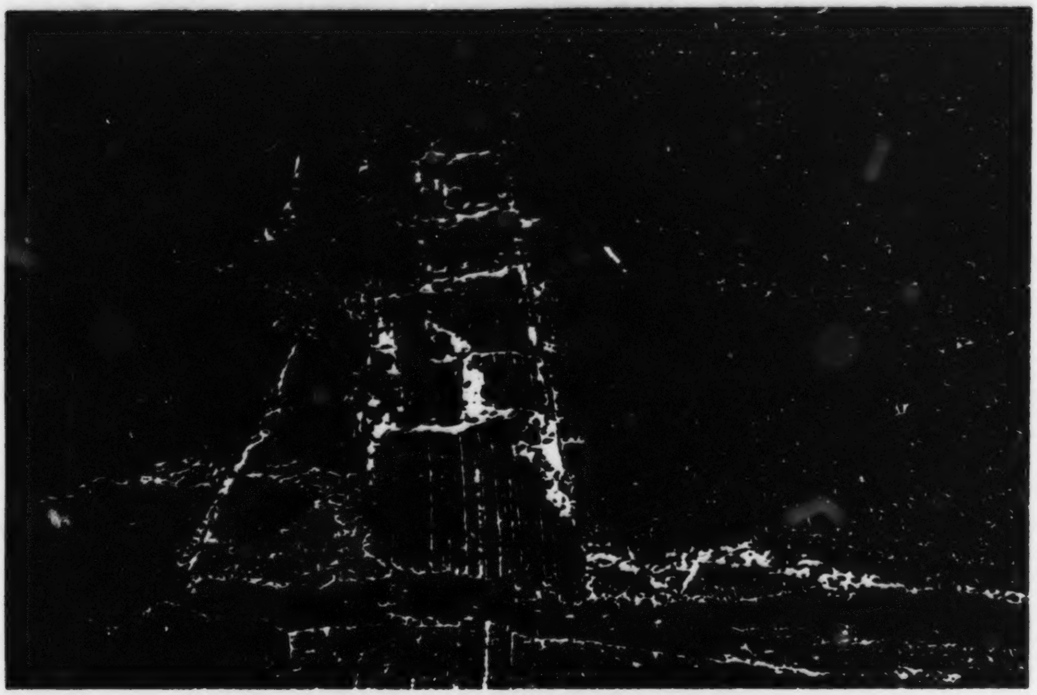


FIG. PBP-7

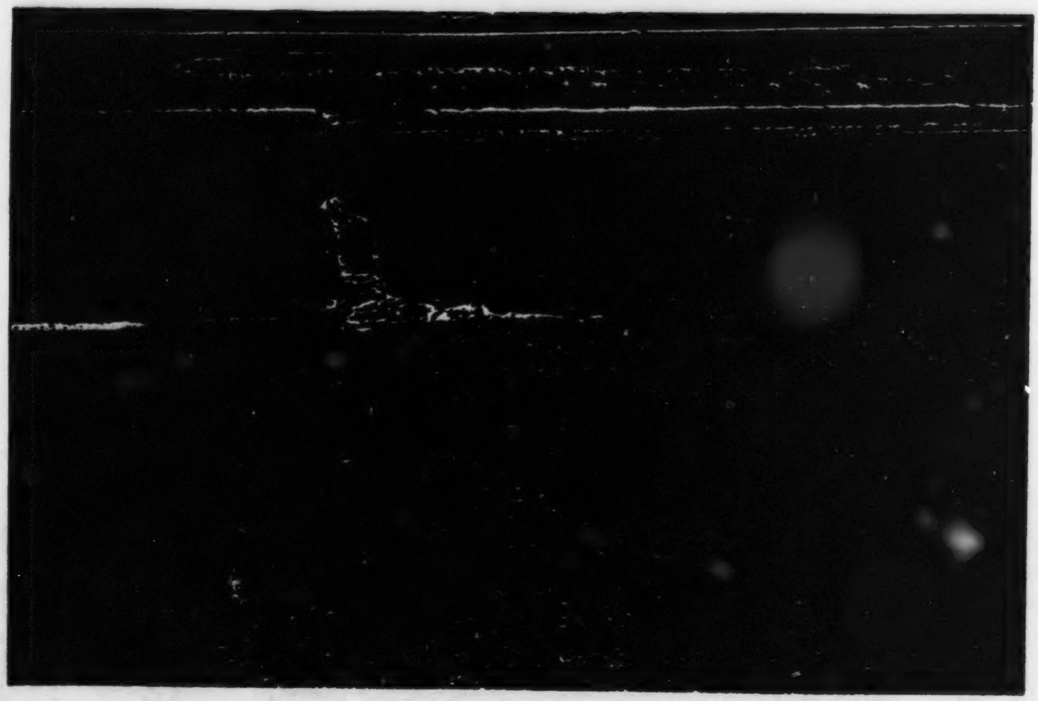


FIG. PBP-8

FIG. PBP-9

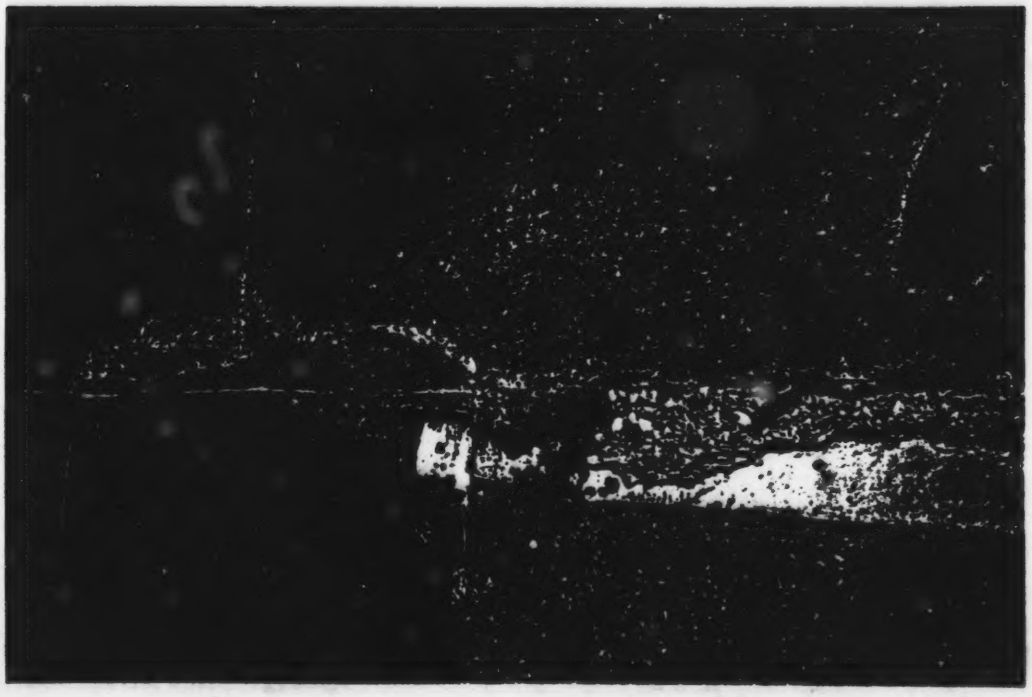


FIG. PBP-10

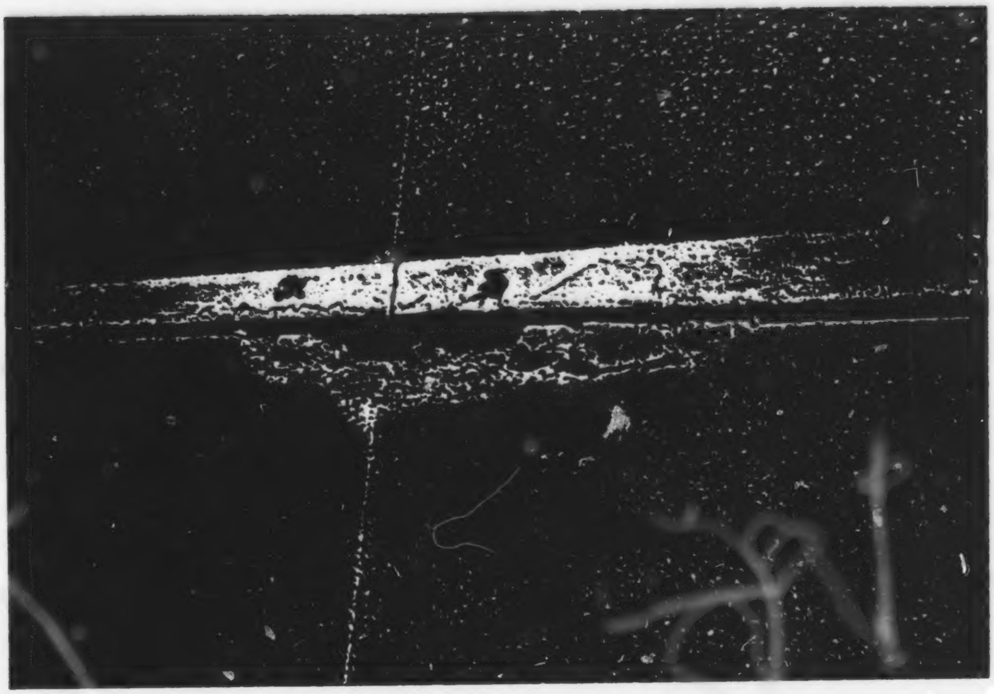


FIG. H59-1

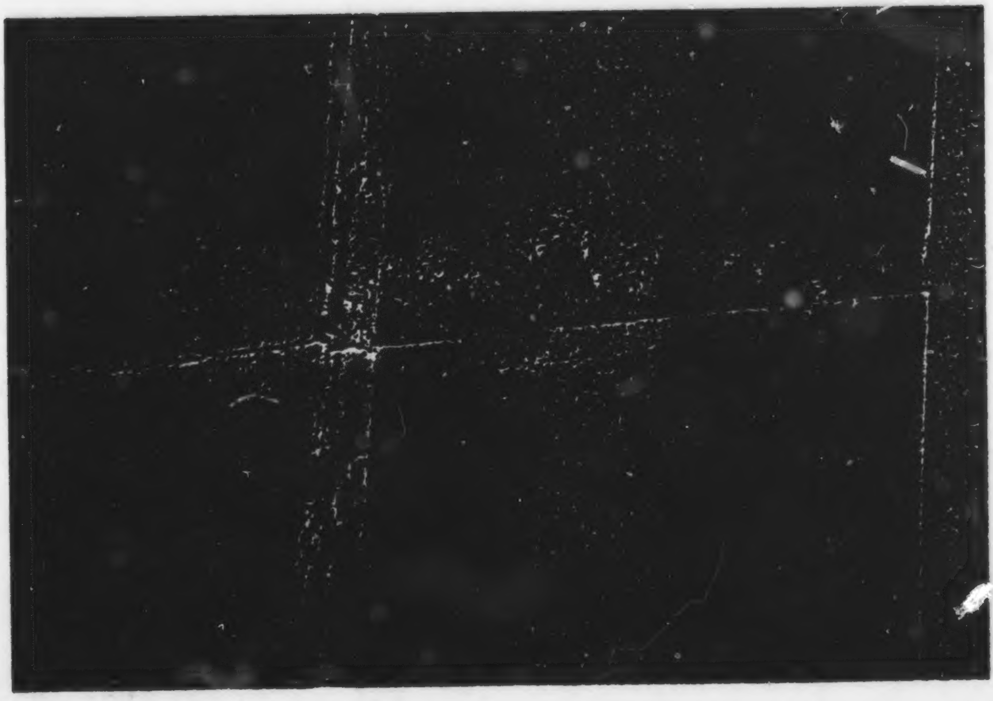


FIG. H59-5

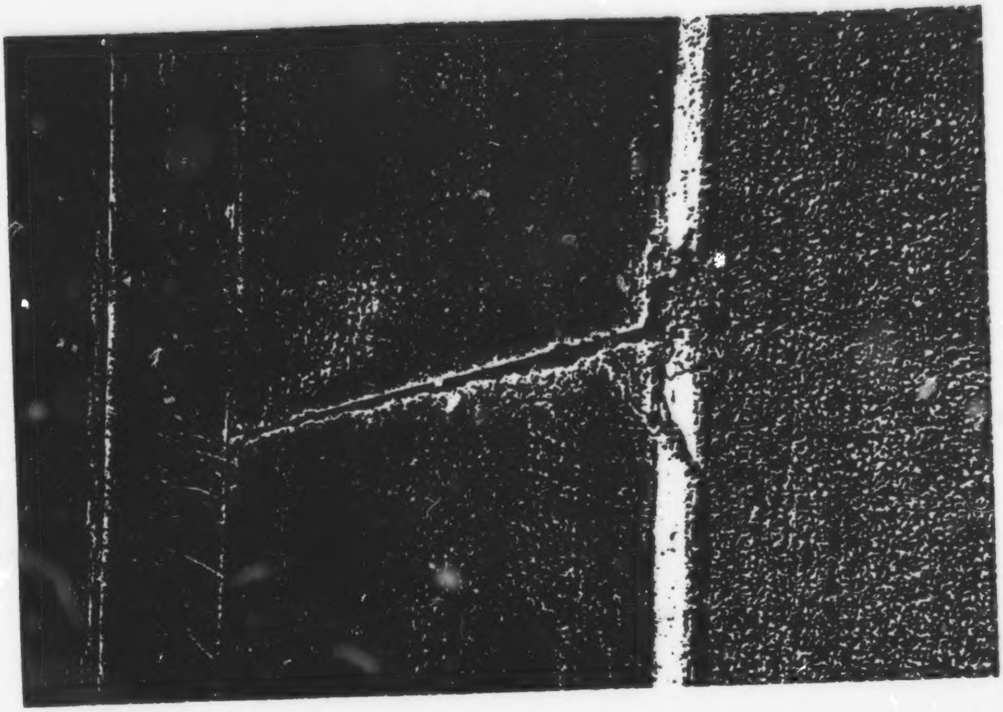


FIG. H59-2

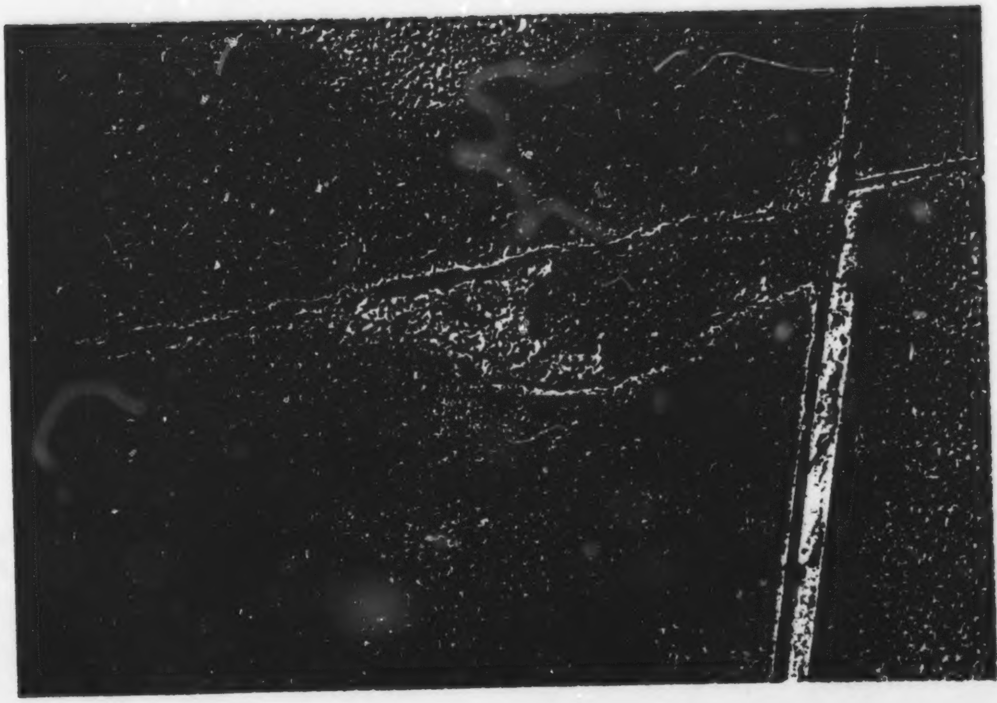


FIG. H59-3

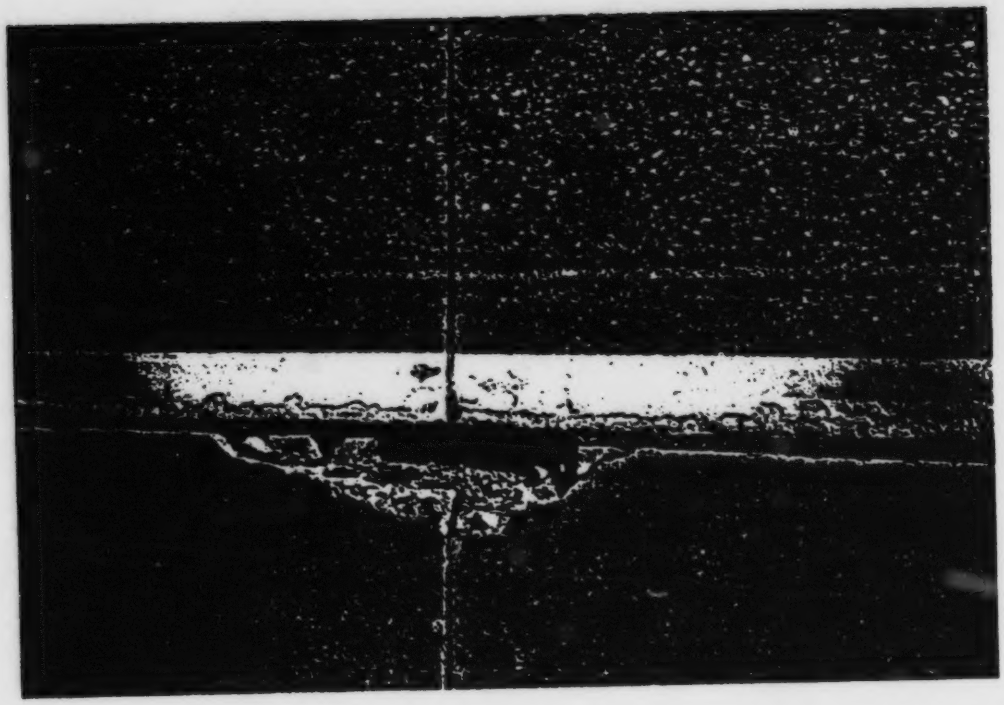


FIG. H59-4

3.5 Observations on D-Cracking of Concrete at Airports in Manitoba:

3.5.1 C.F.B. Winnipeg:

See. Dwg No. SK-W117-0203/1-3.

The apron in front of Hangars 11, 10 and 16 was built in 1951. The portion in front of H16 was replaced in 1980. The maximum aggregate size was 3.8 cm (Area 5). There are a few isolated instances of joint deterioration and replacement due to D-Cracking. A sample survey indicated that 8% of the joints were affected and the average distress severity was 0.27.

The portion in front of H10 was replaced in 1983. The maximum aggregate was downsized to 1.9 cm. There was no evidence of D-Cracking in this section.

The portion in front of H11 had not been replaced and there was extensive and severe D-Cracking at the joints. Most of the "joint intersections" had been replaced. The interior portions of the slabs were generally in good condition.

The parking areas on the north and south sides of the hangars were all constructed in 1951. There was evidence of a fair amount of D-Cracking. A good number of the "joint intersections" or portions thereof, had been replaced. The maximum aggregate size was 3.8 cm.

Area	Constr. Yr.	Occurrence Range	Occurrence Av.	Severity Range	Severity Av.
APRON-RAMP					
Opposite H16	1980		8		0.27
Opposite H10	1983		0		0
Opposite H11	1951	84-95	89	2.10-2.65	2.39
PARKING AREAS					
N side of H11	1951	2-5	4	0.05-0.09	0.07
S side of H11	1951	6-13	10	0.19-0.27	0.23
S side of H10	1951	5-12	8	0.06-0.32	0.19
S side of H16	1951	18-25	22	0.39-0.64	0.52

3.5.2 C.F.B. Portage Airfield:

See plan No. SK-P176-0201-3.

Only apron surveyed. Runways are asphaltic concrete. Did not survey the runway buttons, but, according to the airfield pavement history, the buttons of the main runway were constructed in 1953 and repairs were carried out in 1973.

Many of the "joint intersections" had been replaced (1977), and these are noted by the letter "R" on the survey sheets. They were assigned category 3 in the severity ratings.

With the exception of the joints, the concrete was generally in good condition, indicating that D-Cracking was solely responsible for the deterioration and shortened life.

Area	Constr. Yr.	Occurrence Range	Occurrence Av.	Severity Range	Severity Av.
APRON RAMP					
Opposite H4	1952	76-79	78	1.62-1.79	1.71
Opposite H3	1952	79-82	81	1.82-1.96	1.89
Opposite H1	1952	75-78	76	1.66-1.92	1.79
Opposite Barker	1952	82-86	85	1.73-1.90	1.82
Between Barker and H1	1952	36-53	45	0.73-1.07	0.90

Averages for the Apron-Ramp: Occurrence = 80%; Severity = 1.80

3.5.3 Rivers Airport:

Location	% Occurrence	Severity
EAST-WEST RUNWAY		
Section 1	37	0.37
Section 2	13	0.13
Section 3	18	0.22
Section 4	12	0.13
Section 5	32	0.38
Section 6	25	0.28
Section 7 (East of inters)	97	0.97
Section 8 (East of inters)	88	0.93
Section 9 (East of inters)	80	0.80
SE - NW RUNWAY		
Section 1	97	1.57
Section 2	93	1.65
Section 3	100	1.87
Section 4	100	1.53
Section 5	100	1.97
Section 6	100	1.72
Section 7	90	1.27
Section 8	100	2.10
TAXIWAY		
Section 1	100	1.75
Section 2	100	1.86
Section 3	89	2.07
Section 4	93	1.75
Section 5	100	2.11
Section 6	93	2.07
Section 7	100	1.29
Section 8	100	1.50

This airport has been abandoned for about 14 years. The D-Cracking patterns were very well developed. Because there has been no traffic on the E-W runway, there was no breaking away of the deteriorated concrete and therefore the ratings were far less severe than the ratings would have been had there been traffic.

On the SE-NW runway, the badly deteriorated joints had been replaced. These were given a rating of 3. It appeared that this runway had been used more recently than the E-W runway. Some of the joints appeared to have been replaced fairly recently. The concrete had been broken up by traffic at the joints where D-Cracking was severe.

3.5.4 Churchill Airport:

APRON (abandoned)

Survey No.	No. of Joints	% Affected	Av. Severity
1	80	76	1.55
2	80	84	1.80
3	80	75	1.32
4	56	77	1.59
5	80	68	1.34
6	80	89	1.68
7	80	75	1.51
8	80	81	1.60
9	80	75	1.26
10	80	58	0.80
11	80	52	0.79
12	56	66	1.05
13	80	78	1.25
14	80	67	1.05
15	80	72	0.70
16	80	95	0.98

Total joints = 1,280; Av. Occurrence = 74%; Av. Severity = 1.27.

Note: Because the apron had been abandoned for several years, the D-Cracking patterns were well developed and intact. This resulted in lower severity ratings since the weakened joints had not been broken up by traffic.

TAXIWAY TO RUNWAY 33 FROM ABANDONED APRON

1	80	79	1.60
2	80	88	1.92
3	80	98	2.02
4	80	100	1.72
5	80	88	1.24
6	80	62	0.86

Total joints = 480; Av. Occurrence = 86%; Av. Severity = 1.56.

BUTTON - RUNWAY 25

1	80	100	2.31
2	40	90	1.38
3	80	90	1.35
4	48	100	2.14
5	80	83	1.26
6	80	67	1.05
7	80	95	0.98

Total joints = 488; Av. Occurrence = 89%; Av. Severity = 1.46

PARKING LOT ADJACENT TO TERMINAL

1	56	54	0.95
2	56	55	0.70
3	56	41	0.64
4	168	35	0.62

Total joints = 336; Av. Occurrence = 43%; Av. Severity = 0.70.

BUTTON - RUNWAY 33

All "joint intersections" had been removed and replaced in 1983 because of the concrete deterioration that had taken place.

① 13452
CONCRETE REPAIRS — 1973
229 mm PCC — 1953-54
229 mm BASE COURSE

② 11156
40 mm HMAC — 1987
205 mm HMAC — 1973
406 mm BASE COURSE — 1953

③ 108186
40 mm HMAC — 1987
102 mm HMAC (NOM) — 1973
SLURRY SEAL — 1960
102 mm HMAC — 1953
406 mm BASE COURSE

④ 5574
40 mm HMAC — 1987
102 mm HMAC — 1973
140 mm HMAC — 1966
229 mm GRAN BASE
457 mm GRAN SUBBASE
152 mm SAND MAT

⑤ 14282
40 mm HMAC — 1987
102 mm HMAC (NOM) — 1973
102 mm HMAC — 1953
406 mm BASE COURSE

⑥ 28079
40 mm HMAC — 1987
102 mm HMAC (NOM) — 1973
SLURRY SEAL — 1960
102 mm HMAC — 1953
419 mm BASE COURSE

⑦ 3902
CONCRETE REPAIRS — 1973
229 mm PCC — 1953
309 mm BASE COURSE

⑧ 38462
SLURRY SEAL — 1972
SLURRY SEAL — 1960
102 mm HMAC — 1953
406 mm BASE COURSE

⑨ [illegible]
100 mm HMAC — 1979
SLURRY SEAL — 1972
140 mm HMAC — 1966
229 mm GRAN BASE
457 mm GRAN SUBBASE
152 mm SAND MAT

⑩ 6967
SLURRY SEAL — 1972
140 mm HMAC — 1966
229 mm GRAN SUBBASE
457 mm GRAN SUBBASE
152 mm SAND MAT

⑪ 39298
100 mm HMAC — 1979
SLURRY SEAL — 1972
SLURRY SEAL — 1960
102 mm HMAC — 1953
229 mm BASE COURSE
64 mm HMAC — 1945
457 mm BASE COURSE

⑫ 49120
CONCRETE REPAIRS — 1977
CONCRETE REPAIRS — 12A-1973
229 mm PCC — 1952
152 mm BASE COURSE

⑬ 8995
50 mm HMAC — 1982
CONCRETE REPAIRS — 1979
229 mm PCC — 1952
152 mm BASE COURSE

⑭ 33339
51 mm HMAC — 1971
ASPHALT TREATMENT — 1953
76 mm HMAC — PRE 1945
152 mm BASE COURSE

⑮ [illegible]
[illegible] — 1973
229 mm BASE COURSE
508 mm SUB-BASE COURSE

⑯ 25100
102 mm HMAC — 1973
102 mm HMAC — 1953
152 mm BASE COURSE
279 mm SUBBASE

⑰ 7800
100 mm HMAC — 1979
102 mm HMAC — 1973
102 mm HMAC — 1953
152 mm BASE COURSE
279 mm SUBBASE

⑱ 522
100 mm HMAC — 1979
102 mm HMAC — 1973
SLURRY SEAL — 1960
127 mm HMAC — 1953
229 mm BASE COURSE
63 mm HMAC — PRE-1945
203 mm BASE COURSE

⑲ 2499
100mm HMAC — 1983
150mm BASE COURSE
300mm SUB BASE

⑳ 6542
225 mm PCC — UNDATED
51 mm BASE COURSE

㉑ 564
76 mm HMAC — 1971
152 mm BASE COURSE
102 mm GRAN SUBBASE (NOM)

㉒ [illegible]
[illegible]
229 mm PCC — [illegible]
279 mm BASE COURSE

㉓ 7908
229 mm PCC — 1952
229 mm CR GRAVEL
457 mm GRAVEL BASE

㉔ 1948
229 mm PCC — UNDATED
203 mm BASE COURSE

㉓ 2308
229 mm PCC — 1964
76 mm BASE COURSE
457 mm SUBBASE

㉖ 12192
50 mm HMAC — 1985
229 mm PCC — 1952
254 mm GRAN BASE

㉗ 2680
229 mm PCC — 1953
152 mm PCC — PRE-1945
51 mm BASE COURSE

㉘ 15218
102 mm MIXED IN PLACE ASPHALT 1984
20 mm SURFACE SEAL — 1953
330 mm GRAN BASE

㉙ 16968
250mm PCC — 1982
150mm BASE COURSE
150mm SUB BASE

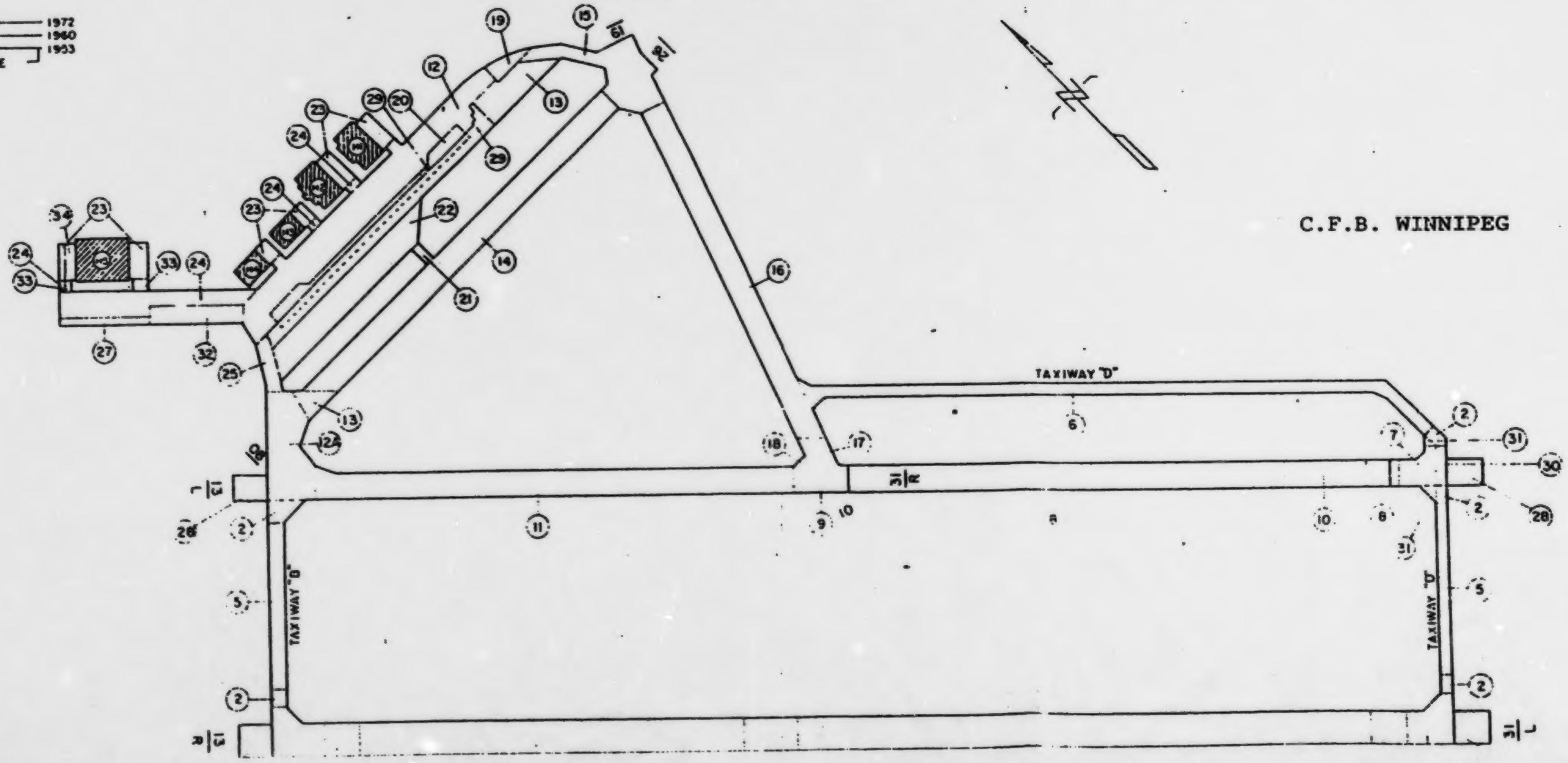
㉜ [illegible]
CONCRETE REPAIRS — 1979
229 mm PCC — 1963
30[illegible]mm BASE COURSE

㉛ 960
203 mm HMAC — 1973
406 mm BASE COURSE

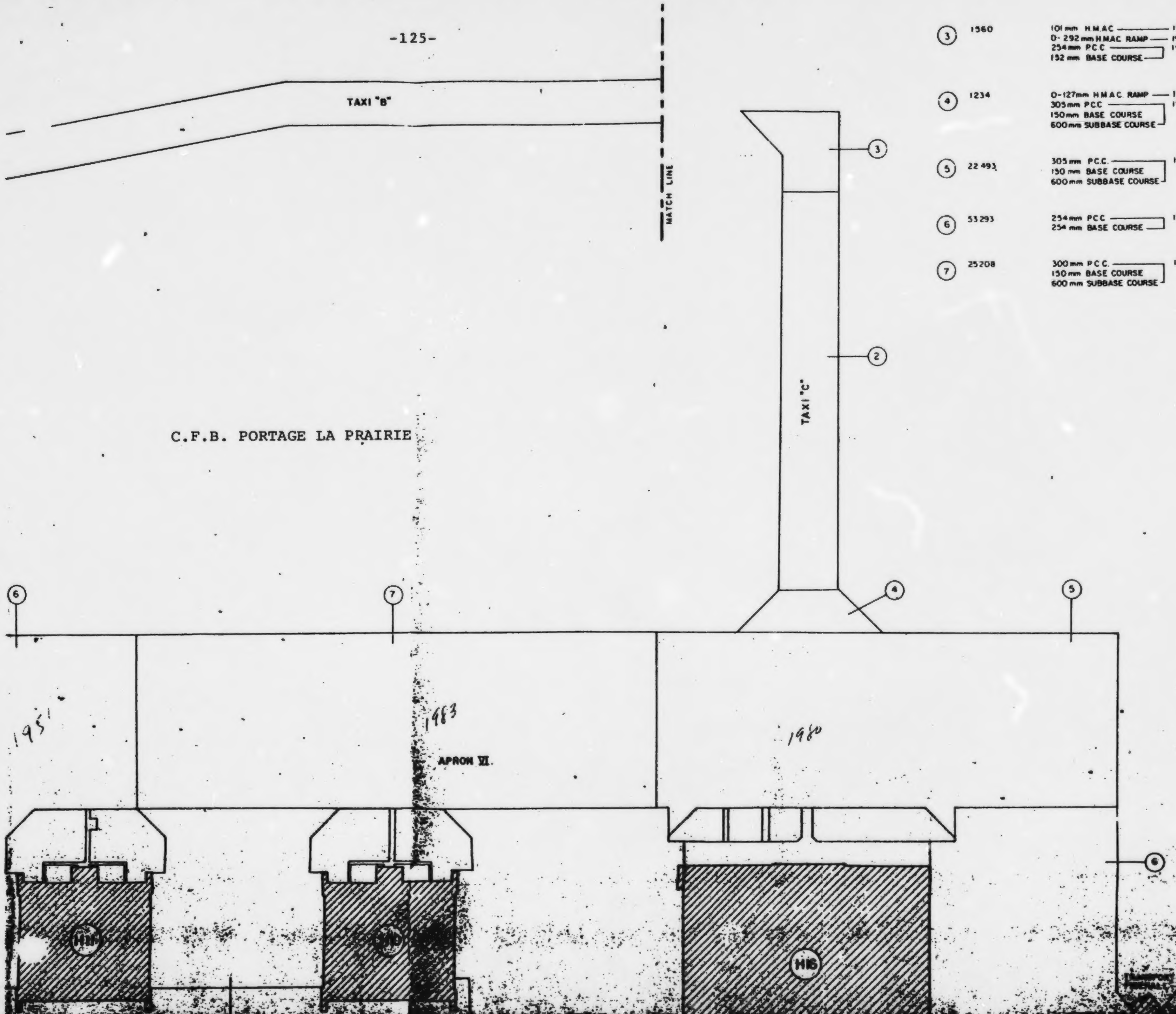
㉚ 5380
230 mm PCC — 1985
150 mm GRAN BASE
150 mm GRAN SUB-BASE

㉝ 560
50 mm HMAC — 1985
229 mm PCC — UNDATED
203 mm BASE COURSE

㉞ 1292
50 mm HMAC — 1985
229 mm PCC — 1952
229 mm CR GRAVEL
457 mm GRAN BASE



C.F.B. WINNIPEG



C.F.B. PORTAGE LA PRAIRIE

③	1560	101 mm H.M.AC 0-292 mm H.M.AC RAMP 254 mm P.C.C 152 mm BASE COURSE	1970 1959 1949
④	1234	0-127mm H.M.A.C. RAMP 305mm PCC 150mm BASE COURSE 600mm SUBBASE COURSE	1985 1980
⑤	22 493	305 mm P.C.C. 150 mm BASE COURSE 600 mm SUBBASE COURSE	1980
⑥	53 293	254 mm PCC 254 mm BASE COURSE	1951
⑦	25208	300 mm P.C.C. 150 mm BASE COURSE 600 mm SUBBASE COURSE	1983

NOTES:

THICKNESS OF PAVEMENT LAYERS SHOWN IS BASED ON CONTRACT DOCUMENT THICKNESS WHERE AVAILABLE.

ABBREVIATION KEY:

H.M.A.C.	HOT MIX ASPHALTIC CONCRETE
P.C.C.	PORTLAND CEMENT CONCRETE
mm	MILLIMETRES

No	DATE	REVISION REVISION	APPR

SCALE - ECHELLE 1:1000
10 0 10 20 30 40 50

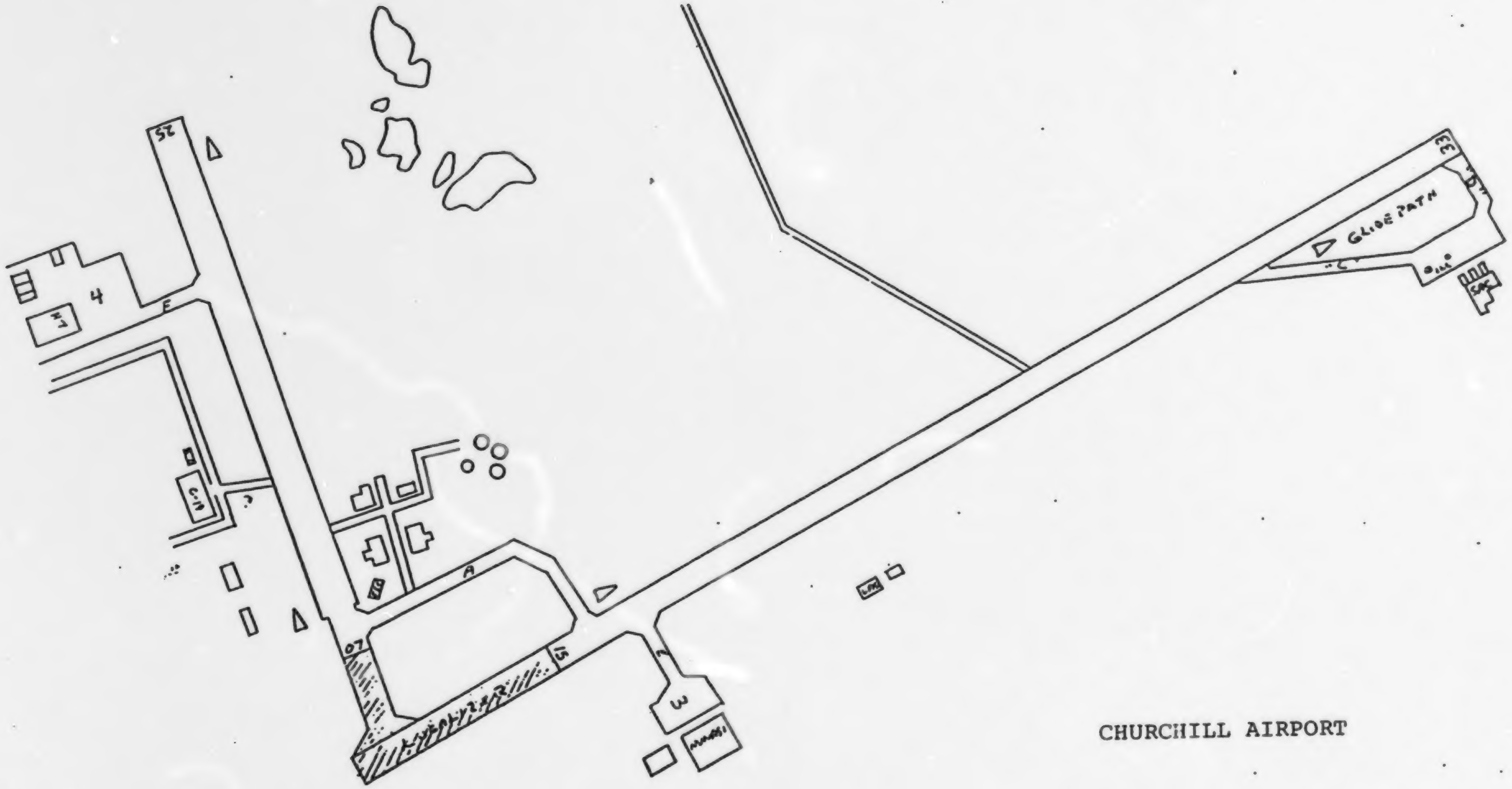
PROJECT -
CFB. WINNIPEG, MANITOBA.

AIRFIELD

CIVIL
DATE 1987-02-27

SUBJECT SUJET
AIRFIELD PAVEMENT HISTORY.

CANADA
PRODUCTION
CONCURRENCE


N
25
4
E
A
07
15
3
33
D
C
GLIDE PATH
SAC

CHURCHILL AIRPORT

3.6 Photographic Descriptions

	DISTRICT 5, CITY OF WINNIPEG
D5- 1	Bentwood Bay from Drake Blvd to Drake Blvd. Shows concrete pavement with multiple structural cracks, severe scaling, extensive popouts, and no evidence of D-Cracking.
D5- 2	Almond Bay of Drake Blvd. Examples of severe scaling and popouts.
D5- 3	As above.
D5- 4	As above.
D5- 5	Arundel Road. Contrast between surface condition of two adjacent lanes.
D5- 6	Berkshire Bay. Photo shows aggregate fracture.
D5- 7	Charing Cross Crescent. Shows the start of D-Cracking in a 9-year old pavement.
D5- 8	St. David. D-Cracking at intersection of structural crack and transverse joint. Generally more severe than at intersection of joints.
D5- 9	As above.
D5-10	As above.
D5-11	St. Louis St. As above.
	RIVERS AIRPORT
R- 1	Well developed D-Cracking pattern with no break-up of concrete because of the absence of traffic. The pavement surface was in very good condition with no signs of scaling, spalling, or popouts. E-W Runway.
R- 2	As above.
R- 3	D-Cracking pattern continuous from joint to joint.
R- 4	SE-NW Runway. Break-up of concrete at joints. Pavement repaired at intersection of longitudinal and transverse joints. These classified as 3 severity.
R- 5	As above.

R- 6	As above.
R- 7	Taxiway. General condition of joints.
R- 8	As above.
R- 9	As above.

HIGHWAY 2

H2- 1	Diagonal cracks at joint intersection (not D-Cracking).
H2- 2	As above.
H2- 6	As above.
H2- 7	As above.
H2- 3	Possible D-Cracking very seldom encountered (2 rating).
H2- 4	Spalling along joints; not classified as D-Cracking.
H2- 5	Repaired joint.

CHURCHILL AIRPORT

CH1 through CH4	Abandoned apron. D-Cracking pattern very well developed. Severe deterioration. Aggregate appears to contain both granitic and carbonate particles. The deterioration in some instances (CH3) included all 4 slabs; whereas in other cases (CH1, 2, and 4), it was limited to 2 slabs. There was no apparent reason for this.
CH5	Taxiway to Runway 33. Deterioration along a longitudinal joint.
CH6	D-Cracking involved all 4 slabs.
CH7	D-Cracking involved 2 slabs only.
CH8 through CH13	Button - Runway 33. Illustrate replacement of concrete at intersection of longitudinal and transverse joints. Note the start of D-Cracking adjacent to the new concrete as indicated by staining and the formation of fine cracks.
CH14	An overall view of Runway 33 which is surfaced with asphaltic concrete.

CH15 & 16	Button - Runway 25. Show very severe D-Cracking at joints.
CH17	Shows break-up of concrete along joints.
CH18	Shows how D-Cracking has progressed throughout the slabs.
CH19 & 20	Show break-up of concrete over entire slab, i.e. the most progressed state of D-Cracking.

DISTRICT 6, CITY OF WINNIPEG

D6- 1,2 & 3	Replacing joints having undergone severe D-Cracking deterioration (along Silverstone, which was constructed in 1965).
D6- 4	Longitudinal and transverse joints which had been replaced along Dalhousie.
D6- 5	Severe D-Cracking along Cornell Drive.
D6- 6	As above.
D6- 7 & 8	Badly fractured pavement with severe scaling and popouts. Corner of Dalhousie and Killarney.
D6- 9	D-Cracking Baldry Bay.
D6-10 & 11	Examples of reinforcement placed too close to surface which probably was the cause of the deterioration.
D6-12 & 13	Photos of joints that had been recently replaced. Dalhousie Street, Fort Richmond.
D6-14 & 15	Joints replaced along Waverly Street, Fort Richmond.
D6-16	Deterioration around manhole Waverly Heights.

C.F.B. PORTAGE

CFBP-1	Example of D-Cracking of a joint that had been replaced in 1973.
CFBP-2	Example of D-Cracking continuing beyond the replaced section of the joint.
CFBP-3	Apron.
CFBP-4, 5 & 6	Other sites on airport.

DISTRICT 3, CITY OF WINNIPEG

D3-1	Structural cracks through asphalt covered concrete. (p. 139).
D3-2	Structural cracks around manhole cover and up road on Scotia Street (no longitudinal joint) (1957). (p. 104).
D3-3	Deterioration on Radford Street (1955). (p. 139).
D3-4	D-Cracking on Sinclair Street (1958). (p. 140).
D3-5	D-Cracking, spalling around manhole cover, Gertrude St. (p. 105).
D3-6	Structural cracking on Gertrude St. (p. 105).
D3-7	D-Cracking. Manhole cover, Egesz St. (1977). (p. 140).
D3-8	Repairs on Selkirk Ave. (1960). (p. 141).
D3-9	Structural cracks -- no longitudinal joint. St. Cross Street (1909). (p. 141).
D3-10	D-Cracking and structural cracks paralleling the longitudinal joint. Margate Road (1974). (p. 102).

DISTRICT 2, CITY OF WINNIPEG

D2-1	Extensive D-Cracking around joint on Assiniboine Ave., Westwood (1966) (p. 142).
D2-2	Scaling of surface finish. Coleridge Park Drive, Westwood. (p. 142).
D2-3	D-Cracking of Daer Blvd. (1941). (p. 143).
D2-4	Spalling of concrete due to reinforcement of too shallow a depth, Vimy Road (1968). (p. 143).
D2-5	D-Cracking, Quail Ridge Rd. (1975). (p. 144).
D2-6	D-Cracking, Leicester Sq. (1957). (p. 144).
D2-7	D-Cracking around manhole covers, Olive St. (p. 102).
D2-8	Spalling of the surface. (p. 104).
D2-8A	D-Cracking and structural cracks. (p. 145).
D2-9	Popouts on Cree Crescent (1971). (p. 146).
D2-10	D-Cracking around manhole covers, Portage Ave. (p. 106).
D2-10A	D-Cracking through asphalt covered concrete on Cavalier Drive (1964). (p. 146).

D2-11	D-Cracking around manhole covers, Portage Ave. (p. 107).
D2-11A	Repairs on Rouge Road. (p. 147).
D2-12	D-Cracking around manhole covers, Portage Ave. (p. 107).

HIGHWAY 7

H7-1	D-Cracking observed on Highway 7. (p. 116).
H7-2	D-Cracking starting at transverse joints. (p. 116).
H7-3 & 4	West twin bridge surface cracking, breakout and plug repairs. (p. 117).
H7-5 & 6	East twin bridge showing scaling, D-Cracking and structural cracking on road surface. (p. 118).

HIGHWAY 8

H8-1	Typical start of D-Cracking. (p. 118A).

PORTAGE LA PRAIRIE BYPASS (GOING WEST)

PBP-1	Longitudinal view of Bypass showing repairs and D-Cracking. (p. 119).
PBP-2	D-Cracking and repairs. (p. 119).
PBP-3	D-Cracking and repairs. (p. 120
PBP-4	D-Cracking and transverse structural cracking. (p. 120).
PBP-5	D-Cracking, repairs and structural cracks. (p. 121).

PORTAGE LA PRAIRIE BYPASS (GOING EAST)

PBP-6	Longitudinal view of Bypass showing D-Cracking at joints. (p. 121).
PBP-7	D-Cracking and structural cracks on repairs. (p. 122).
PBP-8	D-Cracking at intersection and transverse joint. (p. 122).
PBP-9	Longitudinal view showing D-Cracking and repairs. (p. 123).
PBP-10	D-Cracking and repairs along centre-line. (p. 123).

HIGHWAY 59

H59-1	D-Cracking on pavement centre line. (p. 124).
H59-5	D-Cracking and repairs. (p. 124).
H59-2	Repairs and scaling and edge break-out. (p. 125).
H59-3	D-Cracking and repairs. (p. 125).
H59-4	D-Cracking and break-out. (p. 126).

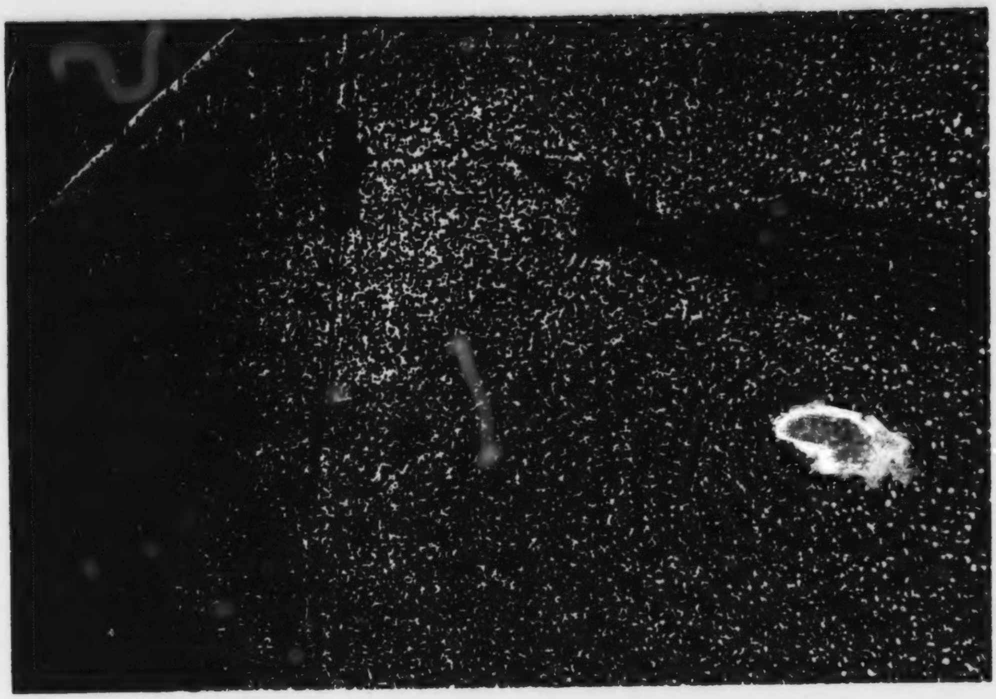


FIG. D3-1

FIG. D3-3

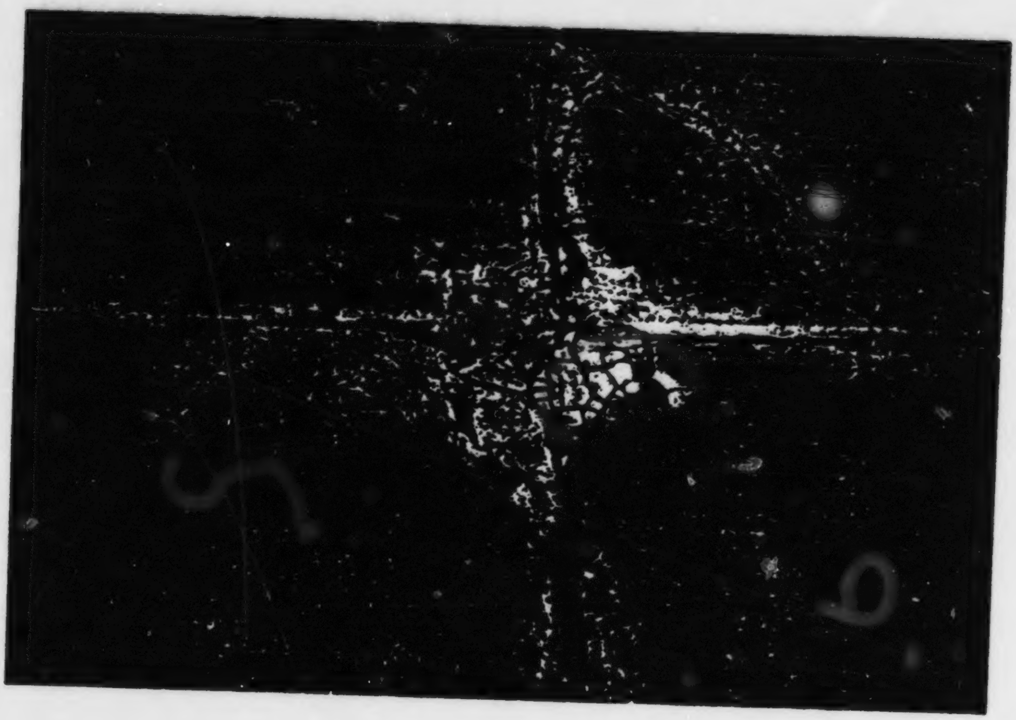


FIG. D3-4

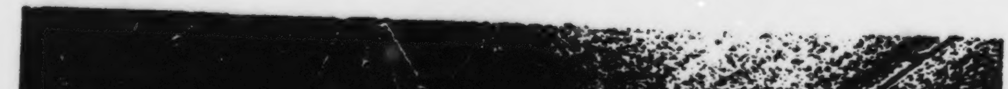


FIG. D3-7

FIG. D3-8

FIG. D3-9

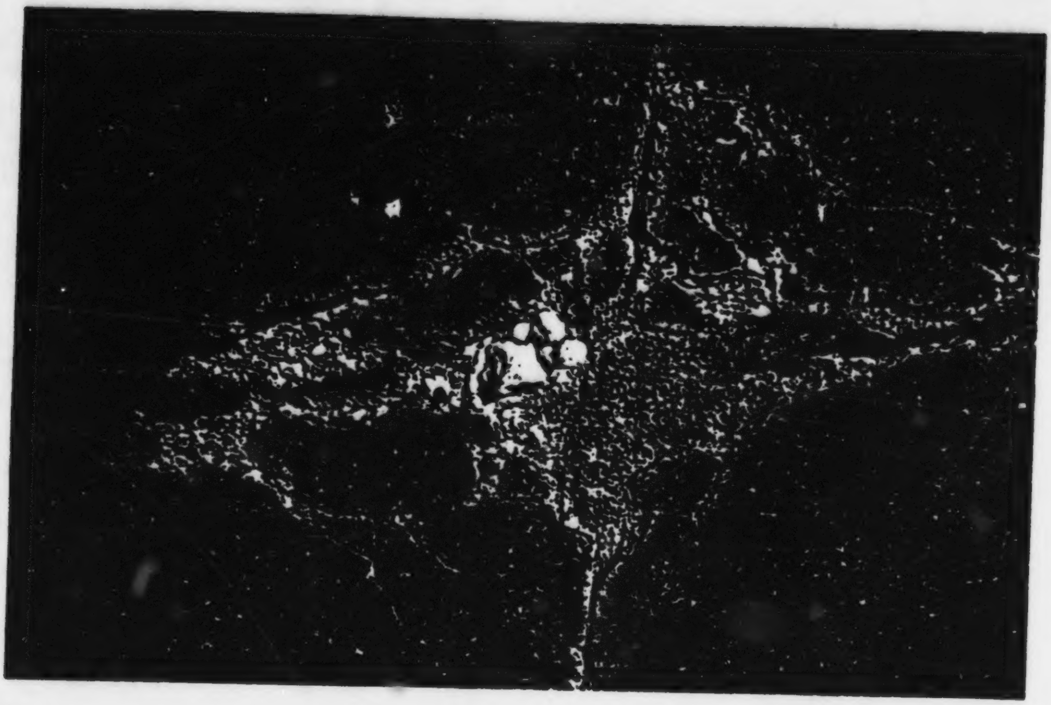


FIG. D2-1

FIG. D2-2

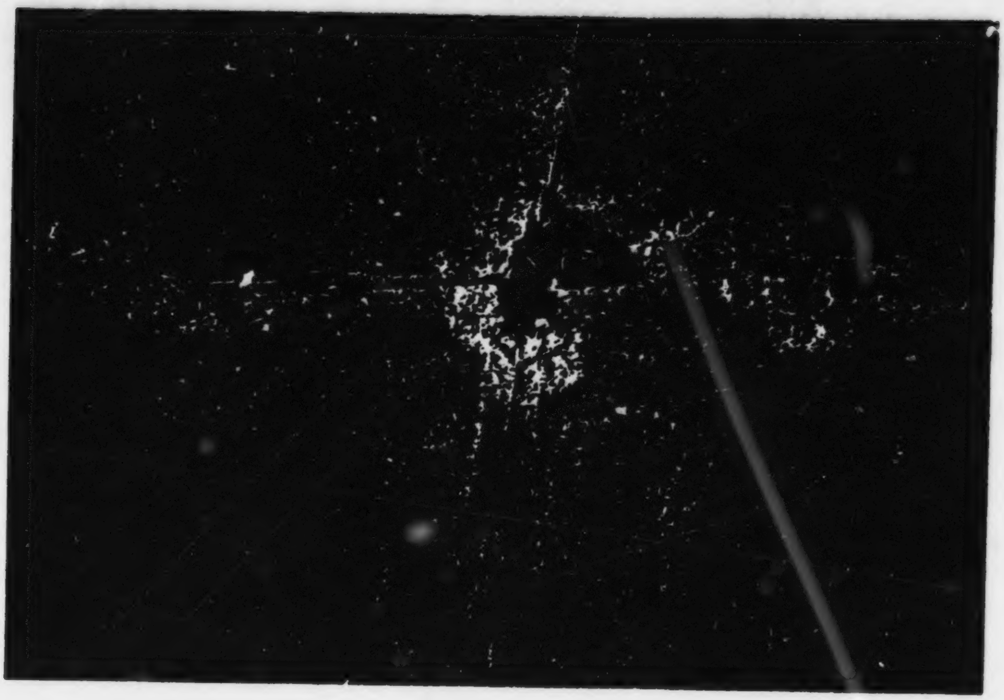


FIG. D2-3

FIG. D2-4

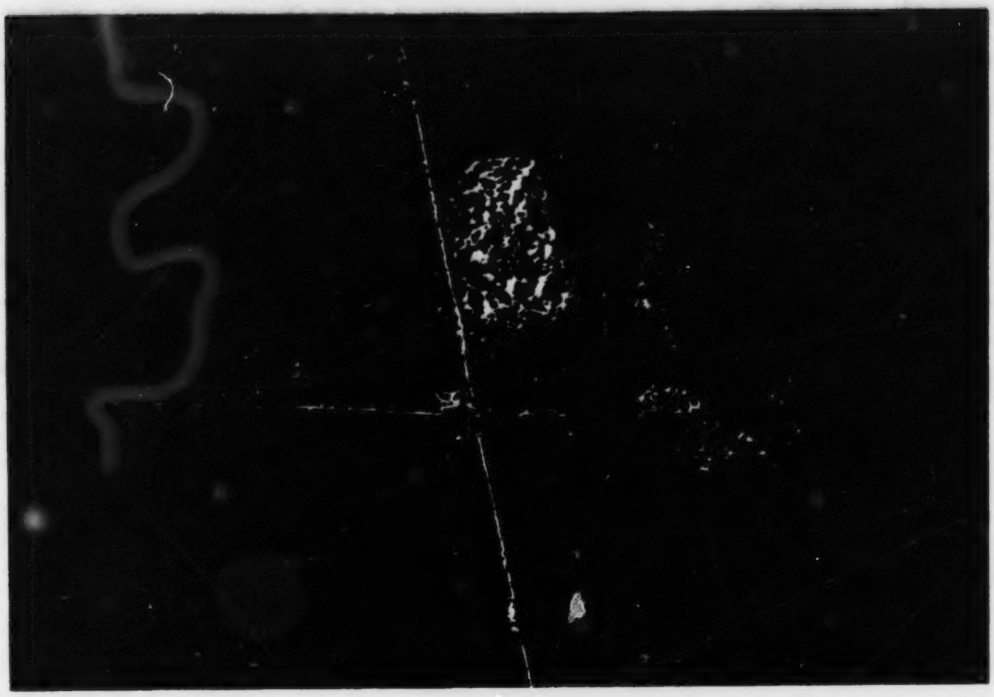


FIG. D2-9

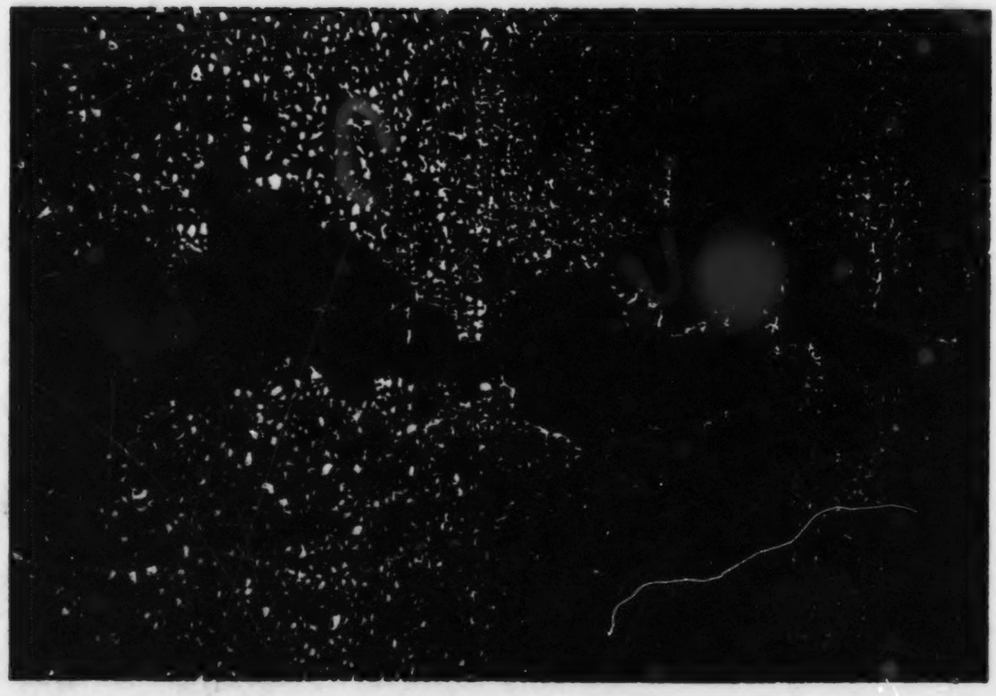


FIG. D2-10A

FIG. D2-11A

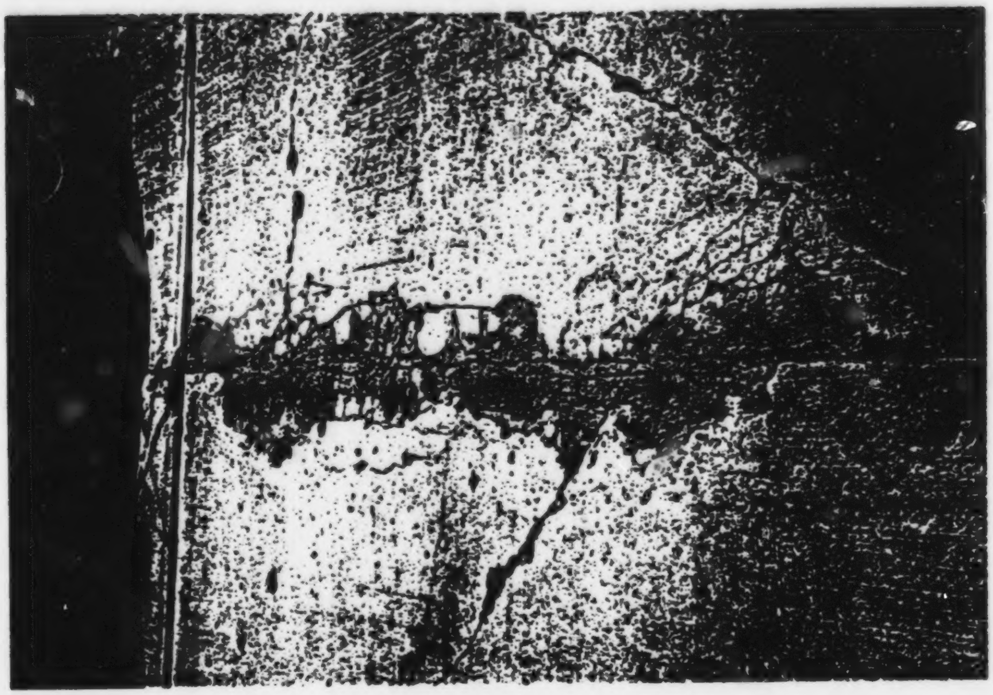


FIG. D2-5

FIG. D2-6

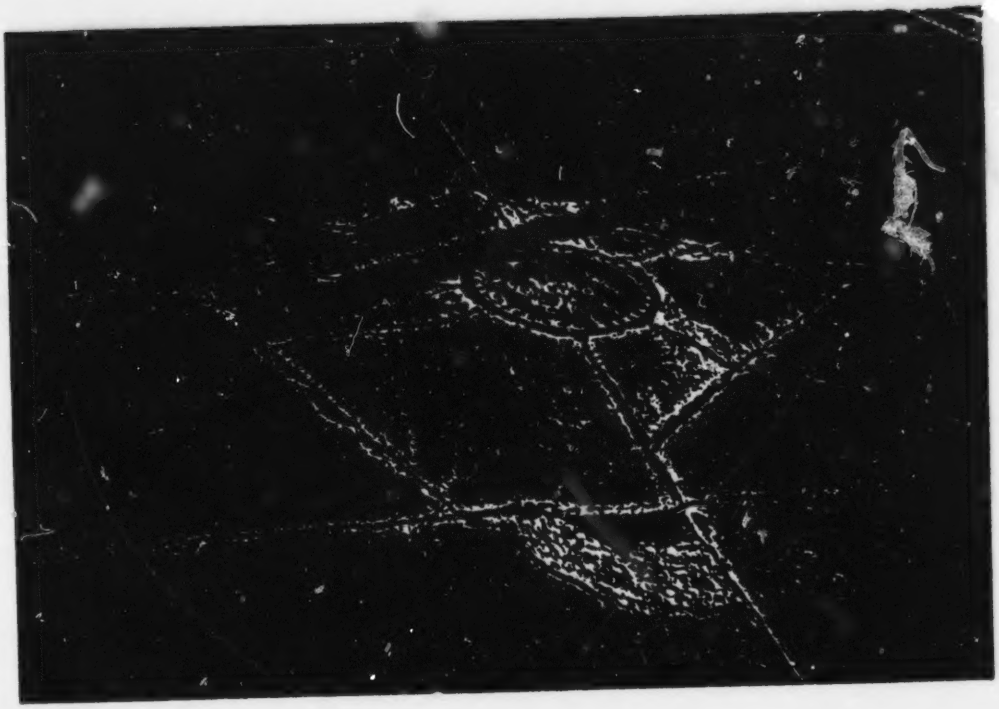


FIG. D2-7

FIG. D2-8A

CHAPTER 4

REMEDIAL MEASURES

The question of remedial measures, as related to the problem of D-Cracking, is separated into two categories: (1) those measures that prevent D-Cracking; and (2) those measures that are intended to prolong the life of an "in service" pavement that is undergoing D-Cracking. Each category is discussed separately in the following sections.

4.1 Measures to Prevent D-Cracking

As noted in Chapter 1, the principal causes of D-Cracking are: the use of certain carbonate coarse aggregates; a freeze-thaw environment; pavement access to external moisture. Since it is virtually impossible to control the latter two causes, prevention can only be accomplished through control of the concrete mixture. In this respect the following measures have been attempted or at least have been considered:

1. Eliminating those aggregates that are known to be susceptible to D-Cracking.

Since it is the carbonate aggregates that have been associated with D-Cracking, eliminating them from concrete pavement construction would appear to be the most logical solution to the problem. However, there are two problems associated with this solution. The obvious one is that there may not be an alternate aggregate, at least from an economic points of view; and, secondly, there still remains the problem of positive identification of bad aggregates, particularly those that are "marginally" acceptable.

2. Limiting the maximum coarse aggregate to a size that has been proven to perform satisfactorily.

The way in which this approach is thought to be beneficial is that the pressures generated both inside and outside the aggregate as a result of freezing, are decreased. The particle size to which the aggregates would have to be reduced would depend on the specific aggregate being used and the environmental conditions at the pavement location. In Manitoba this approach is being implemented to some extend and the following is current practice among the different organizations:

ORGANIZATION	MAXIMUM TOP SIZE
Department of National Defence	20 mm
Transport Canada	20 mm
Department of Highways	
natural gravel	25 mm
quarried limestone	19 mm

However, the practice only started quite recently and there is not enough field data to assess the long term benefits of this approach. Laboratory results of freeze-thaw tests on concrete prisms do support the concept that D-Cracking can be eliminated, or at least controlled, by aggregate size reduction. However, the results of freeze-thaw tests conducted on aggregate alone (Lautridou and Ozoug, 1982; University of Manitoba, unpublished data, Chapter 3) do not unequivocally support this concept.

3. Blending durable and nondurable aggregates.

The intent of this approach is to improve the quality of the concrete by limiting the concentration of deleterious aggregate and thereby keeping their adverse effects to acceptable levels. We believe that, in view of the current limited state of knowledge as to acceptable proportions of durable versus non-durable aggregates, such an approach is not fundamentally sound, and its use should be very limited. In Manitoba there is at least one concrete producer who blends carbonate and igneous aggregates.

4. Separation of durable and nondurable aggregates.

In a very general way, nondurable aggregates tend to be lighter than durable aggregates. This may reflect different mineralogies and also different porosities for aggregates of similar mineralogy. Legg (1956) used a heavy media separation process to obtain high density aggregate. He then added chert particles of various specific gravities and found that the lighter particles led to a decrease in the durability of the concrete. Successful application of this technique was reported by Hanes & Wyman (1962), Kurtz (1962), and Walker & McLaughlin (1965).

5. Improving the concrete durability by air entrainment.

Air entrainment increases the resistance of concrete to frost action by preventing the build up of large pressures within aggregates during the freezing process as explained in Chapter 1. The investigation of air entrainment effects on D-Cracking that was carried out at the University of Manitoba (Section 2.3), showed some promise, but a far more extensive investigation is required before any firm conclusions can be drawn regarding its effects and practicality.

6. Coating aggregates surfaces.

It has been shown by Marek et al (1972), as reported by Thompson et al (1980), that, by heating the aggregates to temperatures in the order of 450° to 800°C, ceramic surfaces can be developed on aggregates whose surface can be cintered or fused. If the mineralogy does not provide for this, fluxes can be added to produce a ceramic surface when the aggregate is heated. The ceramic surfaces are waterproof, insoluble, resistant to chemical attack, and improve the adhesion properties as well as the compressive strength of the aggregate.

According to Marek et al, aggregates can also be coated by thin films of thermosetting and thermoplastic materials which protect them from the elements. The thermoplastic films will, however, reduce the bond strength.

The cost of the above methods of coating aggregates is generally considered to be prohibitive.

7. Impregnating aggregates.

The intent of impregnating aggregates is to drastically reduce their porosity and permeability. Laboratory studies by Kubie et al (1967) showed that linseed oil impregnation improved the durability of concrete substantially, but periodic reapplications were necessary to retain the improved durability.

Polymers have been used for impregnating concrete with beneficial effects, but the writers are not aware of any studies in which the aggregates were impregnated with polymers prior to concrete production.

4.2 Measures for Prolonging the Life of D-Cracked Pavements.

The methods that have been tried in an attempt to prolong the life of a pavement known to be deteriorating as the result of D-Cracking are as follows:

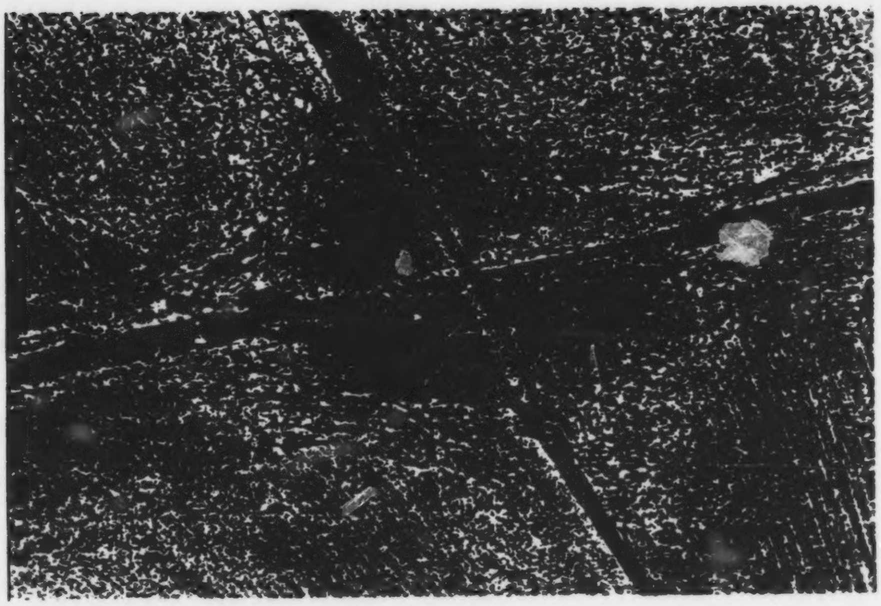
1. Repairing the affected areas.

In most instances D-Cracking starts at the transverse joints of a concrete pavement. The concrete in the vicinity of the joint breaks down, resulting in pot holes and eventually the riding quality of the pavement at the joints becomes unacceptable and the joint must be repaired. Commonly, the loosened aggregate is removed and the pot hole is filled with an asphalt mix or fresh concrete. Examples of joints repairs in this fashion are shown in Figure 4.1. This offers an inexpensive, but a very temporary solution to the problem. The bond between the filler and the concrete soon is destroyed, either by further deterioration of the concrete, or by frost action. This method required periodic replacement of the filler.

2. Removal of the affected concrete and replacement with fresh concrete.

When either the deterioration of a joint becomes so

(a)



(b)

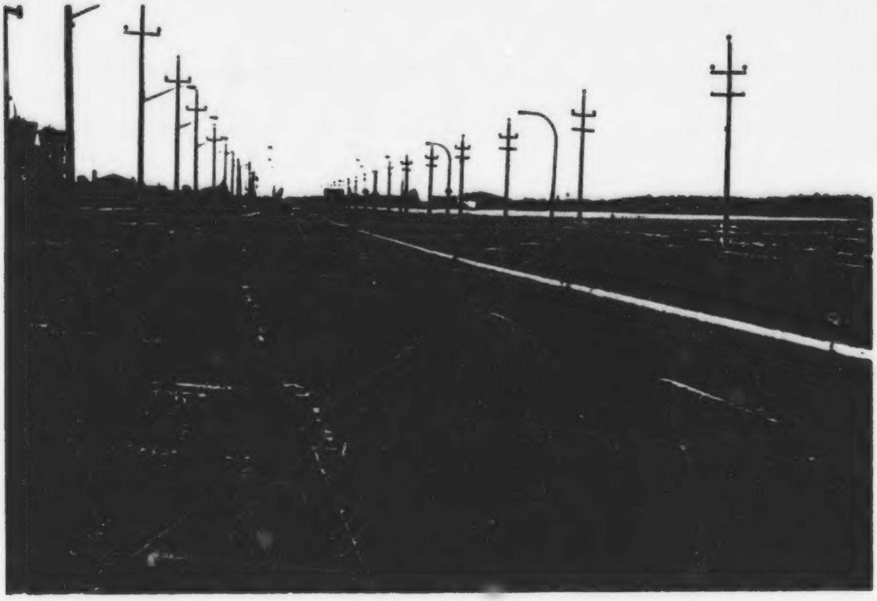
Figure 4.1 Examples of temporary repair at joints (a): asphalt mix, (b) fresh concrete

severe that patching is no longer satisfactory, or if a great percentage of the joints have been seriously affected, the joints or portions of the joints, are removed and replaced with either a hot asphalt mix or fresh concrete. This is a practice that has been adopted by the City of Winnipeg, the Provincial Department of Highways, Transport Canada, and the Defence Department. Photographs of joints that have been repaired on city streets, on provincial highways, and on airport runways in this manner, are shown in Figures 4.2 through 4.4. Typically, a strip of concrete, approximately 2m in width, is removed and replaced at the affected joint. Joints being repaired in this manner are shown in Figure 4.5.

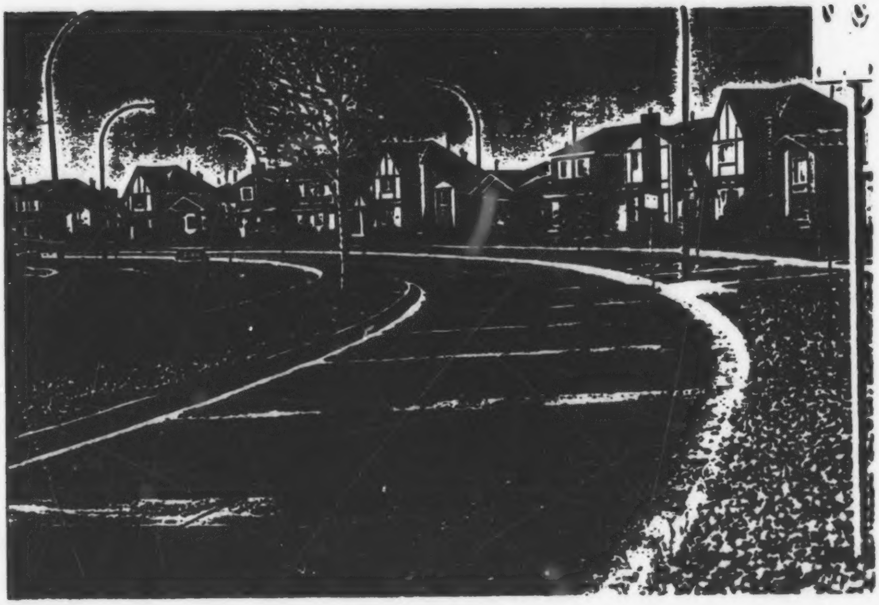
This method does prolong the life of the pavement, but does not solve the problem entirely. In time, D-Cracking starts at the new joint between the fresh and old concrete, as shown in the photos of Figure 4.6. In some instances it was observed that D-Cracking of the fresh concrete had also started, as illustrated in Figure 4.7.

The cost of this method is quite high since it involves costly equipment for breaking up and removing the affected concrete and it requires a considerable number of man hours to prepare the area and place the fresh concrete. As an example, the cost of repairing a major street in Winnipeg in this manner, including an asphalt overlay, was approximately $116,000 per lane per km in 1988. The estimated cost of removing existing concrete and replacing it with new concrete was about $274,000 per lane per km.

The economics of repair versus replacement is difficult to



D6-33

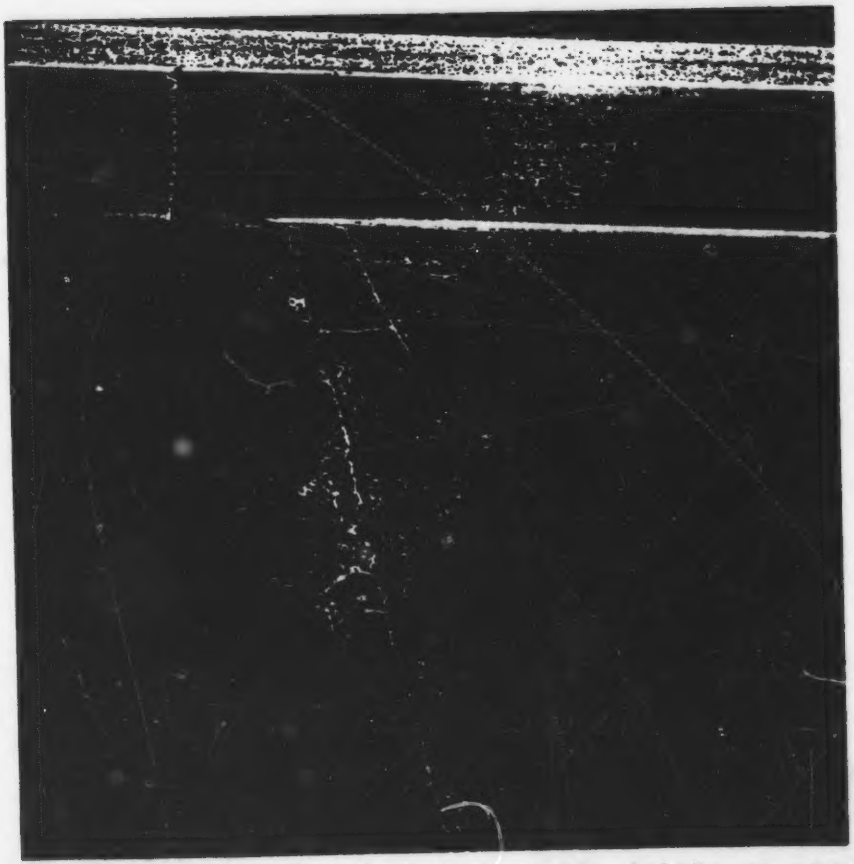


D6-30

Figure 4.2 Concrete replaced at transverse joints, Winnipeg City streets

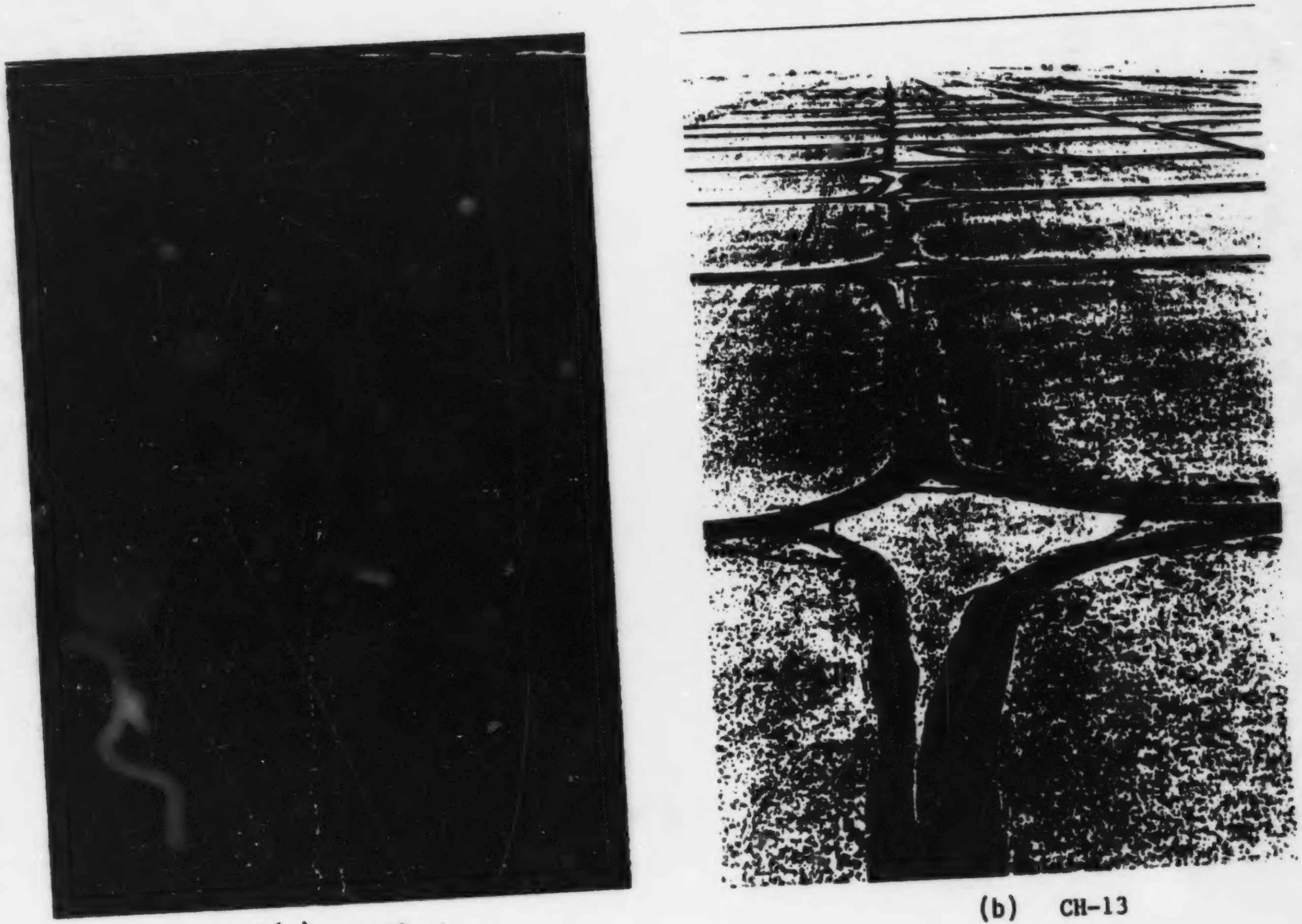
H2-25

(a)



(b)

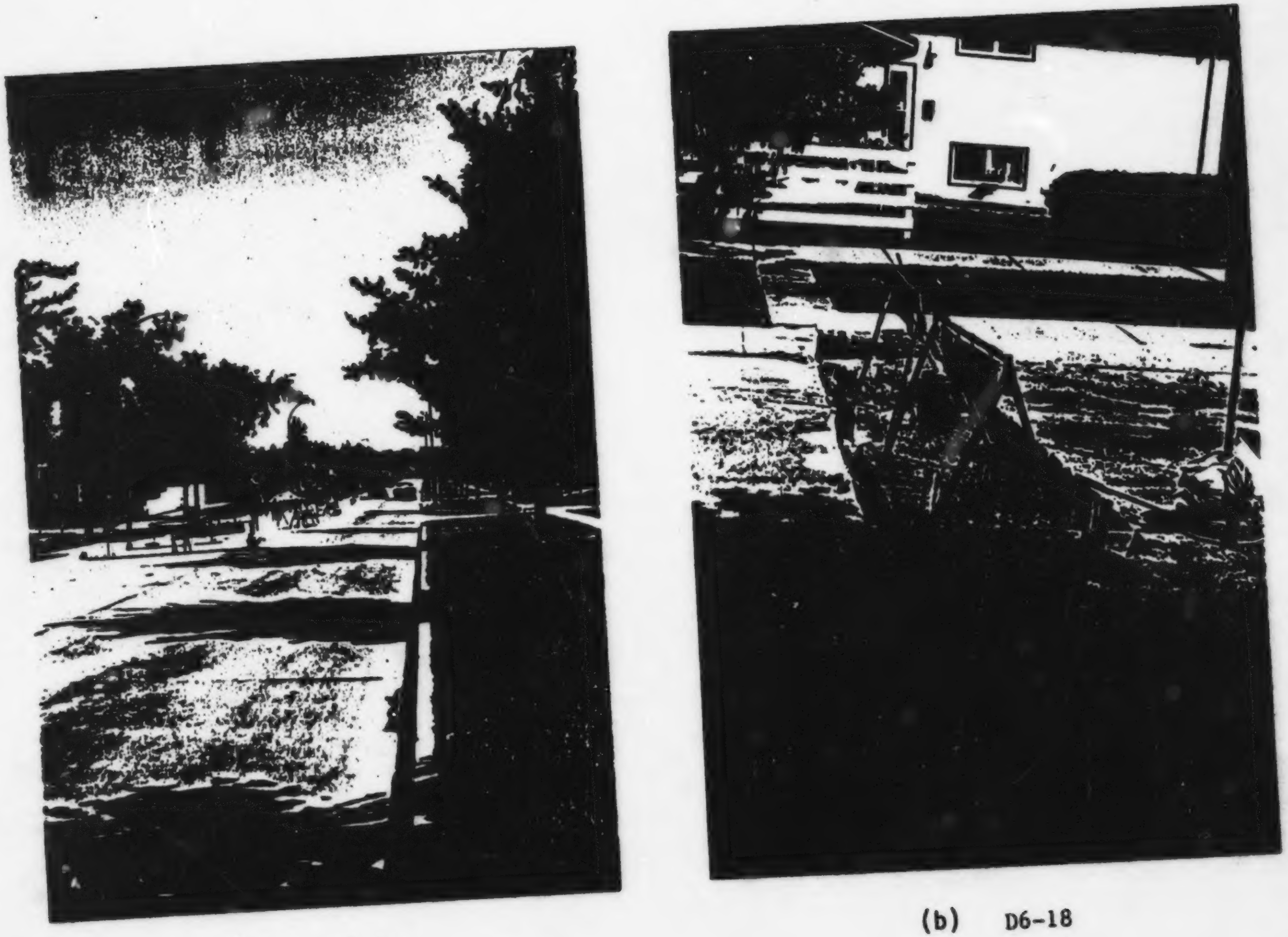
Figure 4.3 Concrete replaced at transverse joints on provincial highways (a) Hwy No. 2, (b) Hwy No. 1W



(a) R-10

(b) CH-13

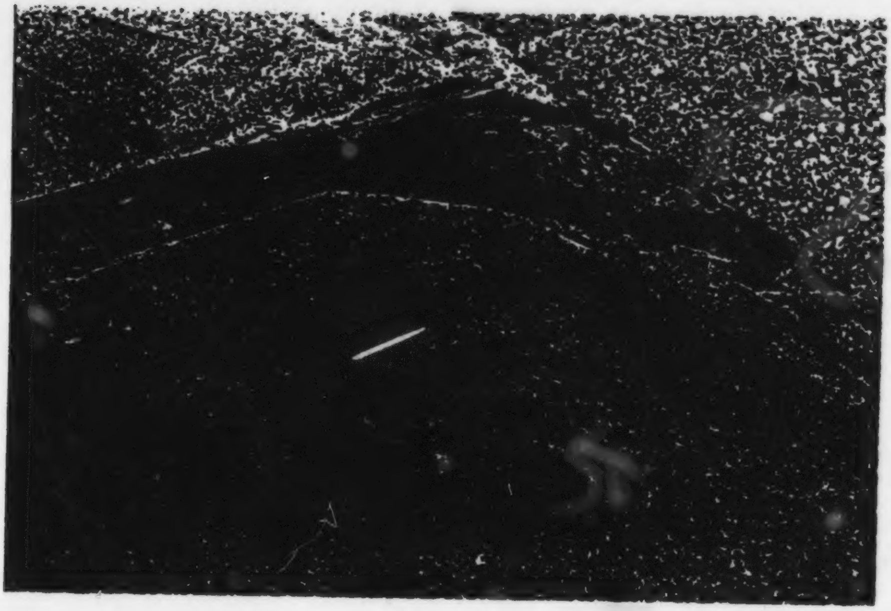
Figure 4.4 Repair at runway joints with fresh concrete
(a) Rivers airfield, (b) Churchill airfield



(a) D6-17

(b) D6-18

Figure 4.5 Replacing concrete of transverse joints on a Winnipeg city street
(a) general view, (b) close-up



CH-17

Figure 4.6 D-Cracking extending beyond the repaired joint

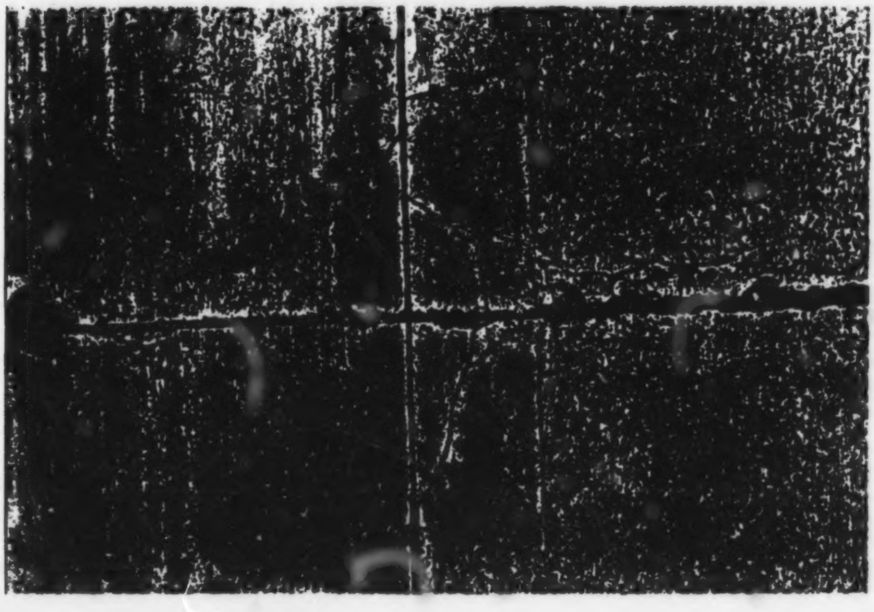


Figure 4.7 D-Cracking of new concrete

evaluate because, at this time, there are no data available as to what the extended life of the "repaired" pavement will be.

3. Applying an asphaltic concrete overlay to the affected pavement.

The application of an asphaltic concrete overlay will certainly improve the riding quality of the pavement, temporarily at least, but little was known about the effect of the overlay on the deterioration process until an extensive study was carried out at the University of Illinois (Janssen & Dempsey, 1985). The following is an excerpt from the abstract of their publication:

"Laboratory freeze-thaw durability tests duplicating field conditions for Interstate 70 near Vandalia, Illinois were conducted. The laboratory PCC [Portland Cement Concrete] samples were made with the same coarse aggregate as the D-Cracked I 70 pavement section. The samples had either no overlay, or 2 inch, 4 inch, or 6 inch overlays. A freeze-thaw cycle determined from actual climatic data was used to simulate winter freeze-thaw cycling. All of the PCC samples, cycled to the equivalent of 5 years of winter exposure, showed strength loss as determined by split tensile tests. The samples with 4 inch overlays showed the most strength loss.

It was concluded that AC [Asphaltic Concrete] overlays do not prevent the progression of D-Cracking in PCC; instead, some overlay thicknesses accelerate the deterioration. When AC overlays are designed for D-Cracked PCC pavements, the effect of decreasing strength of the deteriorating PCC should be considered."

The results of the split tensile tests, taken from their publication, are shown in Figures 4.8 and 4.9. The fact that the strength loss was influenced by the thickness of the AC overlay is not surprising given that the magnitude of pressures generated during the freezing process depends on the absolute value of the temperature and the thermal gradients, both of which are influenced by the thickness of the AC overlay.

The primary value of the tests results is that they indicate that the use of an AC overlay will not terminate the D-Cracking process. Whether or not the overlay will prolong the useful life of the pavement was not answered since it is quite possible that, despite a somewhat larger loss in strength (25% for 4 inch overlay vs 19% for no overlay), the overlay would prevent ravelling and the development of pot holes, thereby increasing the pavement's useful life.

4. Impregnation of concrete.

The idea of halting the deterioration of concrete by impregnating it with some substance, has been investigated to some extent. Kubie et al (1967) found that application of linseed oil after deterioration had already started prevented further deterioration from occurring. One advantage of using linseed oil is that it can be applied with conventional sprayers. Polymer impregnation has also been investigated. Such concrete has increased freeze-thaw durability, higher compressive strength, and enhanced resistance to sulfate attack and scaling (Dikeou et al, 1969). To achieve a "polymer impregnated concrete", (PIC), cured concrete is impregnated with a monomer which is then

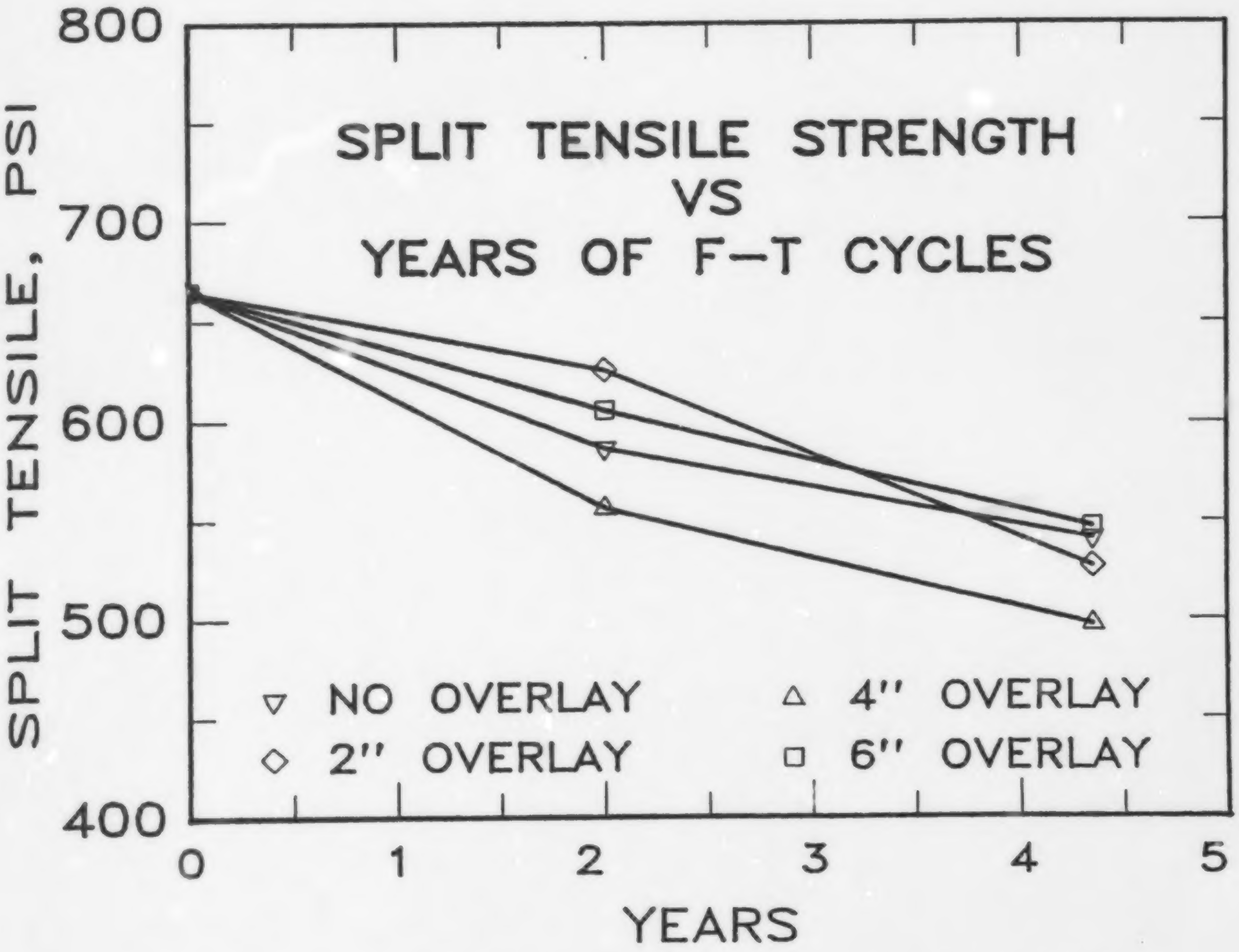


Figure 4.8 Split Tensile Strength versus Years of Freeze-Thaw
(after Janssen & Dempsey)

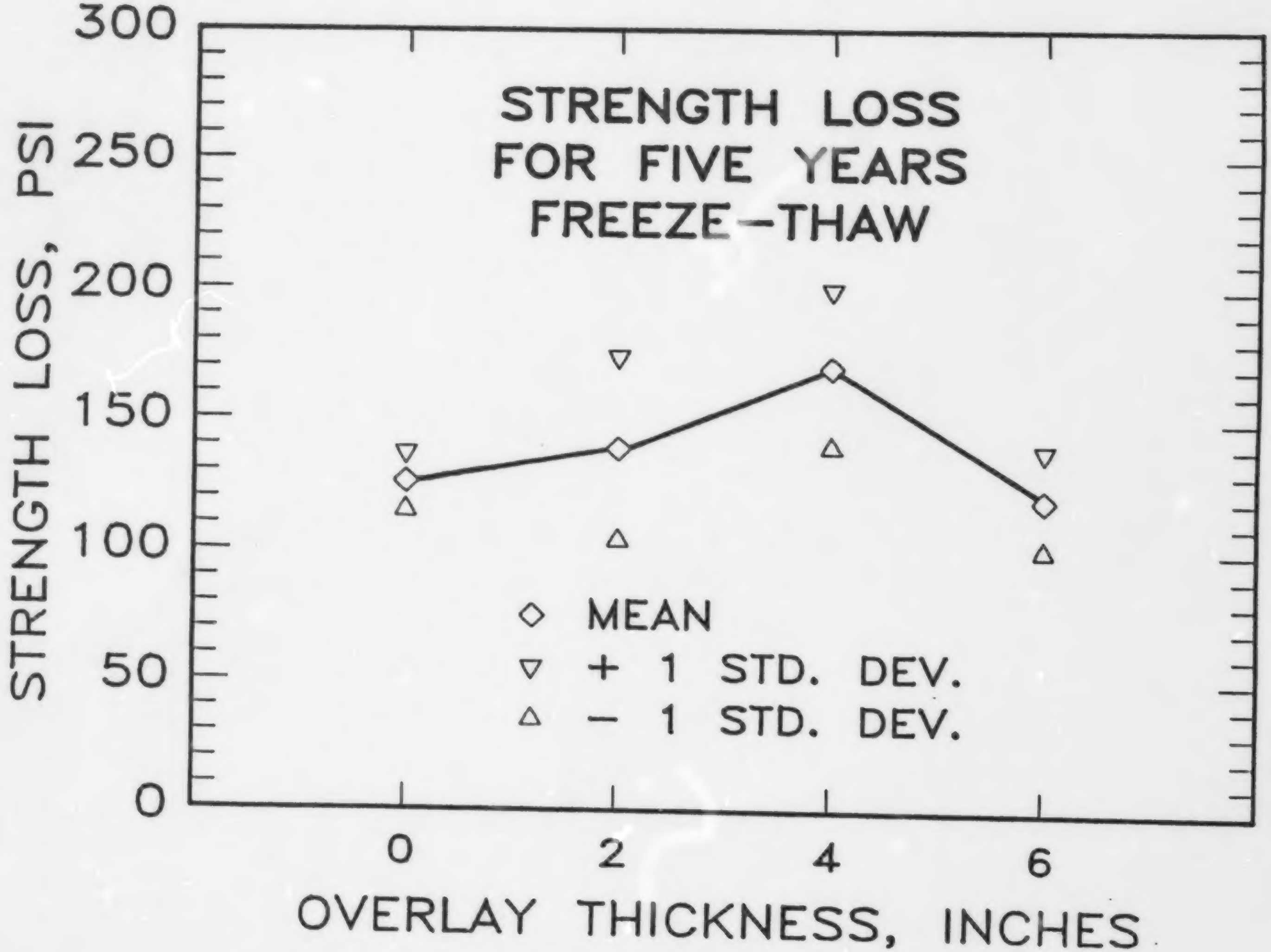


Figure 4.9 Strength Loss versus Overlay Thickness, 5-Year Samples (after Janssen & Dempsey)

polymerized in situ. For penetration of the monomer, the concrete must be thoroughly dried, which requires that a temperature of at least 110°C can be attained. For field applications, Bartholomew et al (1978) found that the temperature of the surface of the slab had to be raised to about 150°C in order to achieve a penetration of 25 mm. Manson et al (1975) found that, in order to achieve a penetration of about 100 mm, a surface temperature in excess of 300°C was required.

After the concrete slab is dried, the monomer is pooled on the slab surface and allowed to soak into the concrete. The monomer is then polymerized thermocatalitically by reheating to 50-80°C for approximately 12 hours.

Since impregnation has been shown to be practical and effective, its potential as a means of extending the useful life of D-Cracked concrete should be given further consideration.

The question of prolonging the life of pavements which are in the process of undergoing D-Cracking is of vital importance to Manitoba, given the high percentage of pavements currently affected by this form of deterioration.

CHAPTER 5

RECOMMENDATIONS FOR FURTHER RESEARCH

5.1 INTRODUCTION

Given that (1) the problem of D-Cracking of concrete pavements is very prevalent in the province of Manitoba (and in all likelihood in other Canadian provinces where carbonate aggregates are used); (2) the costs associated with maintenance and repair of pavements suffering from D-Cracking are high; (3) the useful life of such pavements is drastically reduced; and (4) a practical and economical solution to the problem has not as yet been found, it follows that further research is required.

The research needs may be categorized as:

1. Those associated with producing a concrete that is not susceptible to D-Cracking, and

2. Those that will serve to prolong the useful life of existing pavements that have been affected.

The research needs are listed according to the above categories, in the sections that follow.

5.2 Producing Concrete that is Nonsusceptible to D-Cracking:

1. A practical inexpensive method for identifying aggregates that are susceptible to D-Cracking.

There is a need for a test to replace the ASTM C-666 B test which is currently used. A test in which the aggregate, rather than concrete prisms, is tested would be preferable. The vast amount of information and data on frost shattering that the French have acquired at their CNRS Centre de Geomorphologie should be examined thoroughly to determine its relevance and applicability to the D-Cracking problem.

2. A detailed field and laboratory study of pavements in which the coarse aggregate top-size was reduced.

Reduction of aggregate top size is potentially the most practical and least expensive method of prolonging the life of concrete pavements. Unfortunately, at present, in Manitoba at least, there are no long term data on the performance of such pavements. Since the question of whether or not the life is significantly prolonged is of immediate concern, it is recommended that cores be recovered from these pavements and subjected to an accelerated freeze-thaw environment in a laboratory, in an attempt to assess the pavements' long term performance. As well, a more exhaustive study of the performance of pavements (with reduced top size of coarse aggregate) in other regions of Canada and the United States should be undertaken.

3. Use of air entrainment to eliminate D-Cracking.

As mentioned in an earlier chapter, a very limited study indicated that air entrainment, in relatively large amounts, was effective in reducing the susceptibility of the concrete to D-Cracking. However, the inordinately large amount required may have undesirable effects on other properties of the concrete. This could only be determined by a more comprehensive investigation.

5.3 Prolonging the Life of Existing Pavements.

The use of polymers to halt the D-Cracking process warrants further investigation. This method has been shown to be effective, but it requires further study to improve the technique. The method of achieving impregnation of the monomer requires improvement as this seems to be the most difficult and costly phase of the process. If the need for drying out the pavement could be eliminated, the method would be greatly enhanced.

Although just about every facet of D-Cracking warrants further research, we believe that the above items would provide the most immediate benefits.

5.4 The suggestions for research up to this point have come from a civil engineering viewpoint by Dr. Len Domaschuk. The following is a further proposal for research originating from INRAD from a chemical engineering viewpoint.

The one element in common that D-Cracking defects have is the pavement separation, or crack, on which the crack-out originates. This crack is usually the high point or ridge on a road's cross-section. The similarity between the road's cross-sectional shape and that of a hip roof on a house is obvious. On a house, however, a cap is placed over the joint where the planes of the roof meet. Without this cap, moisture would penetrate at the roof joint and the house would experience considerable deterioration from water damage.

White urethane strips have been adhered to pavement to mark the centre-line of roads and crosswalks. Urethane withstands abuse and is resilient. The polyether based urethanes have shown superior hydrolytic stability compared to ester-based urethanes. Urethanes show outstanding tear resistance, abrasion resistance, ozone/oxidation resistance, and excellent sunlight aging. Heat resistance is up to 107°C and low temperature flexibility down to -62°C. The polyether urethanes are resistance to oils and aliphatic gasolines. They showed less resistance to aromatic gasolines when the samples are totally immersed for 10 days at 21°C. Rating was by change of volume.

Our proposal therefore is to apply a white T-shaped centre strip, designed to serve as a cap between slabs, the stem to be inserted in the centre crack, with urethane adhesive sealing the top flanges to each slab, the top centre of the T to be raised to facilitate run-off. The lateral cracks between slabs to be covered in a similar manner with a natural or transparent strip adhered at the upper end to the white centre strip, thus sealing all the open cracks present to prevent water access from the top of the slab. The stem of the T will be squeezable to ensure a tight fit in the crack and to allow application by machine.

It has been shown that preventing water rising from below the slab by placing a polyethylene vapour barrier between the concrete and the sub-base promoted D-Cracking (Stark, 1970; Girard et al, 1982). This test appears to show that moisture from above the slab which cannot readily drain away, promotes D-Cracking.

The statement on page 11, Chapter 1: "The question of whether or not access to external moisture is an essential requirement, or simply an influential factor, has not been ascertained unequivocally" could be readily confirmed by treating new concrete slabs with 15 mils* of a heavily filled urethane which has been proven, according to Doyle, to be sufficient to adequately protect for many years. Older structures already beginning to crack and spall will require extensive repair work, plus topping up to ¼" thick. Because of the low cost of fillers in the polymer, such toppings are reportedly less expensive than they may at first appears: the polymer may form only 10% of the total.

SUBMITTED BY:
INRAD INDUSTRIAL RESEARCH & DEVELOPMENT LTD.

Per: Edward A. Speers

Edward A. Speers, P.Eng.
President

PROVINCE OF MANITOBA
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SPEERS
REGISTERED ENGINEER

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* mil = 0.001" (0.254 mm)

REFERENCES

Alford, N.M. and A.A. Raham. "An Assessment of Porosity and Pore-Sizes in Hardened Cement Paste". Jr. Mat. Science 16, 1981, pp. 3105-3114.

Axon, E.O., L.T. Murray and R.M. Rucker (1986). "Laboratory Freeze-Thaw Tests vs Outdoor Exposure Tests". HRR 268, Highway Research Board, Washington, D.C. pp. 35-47.

Berry, E.E. and V.M. Malhotra. "Fly Ash in Concrete: A Critical Review". ACI Journal, pp. 59-73. (1980).

Bukovatz, J.E., C.F. Crumpton and H.E. Worley. "Study of D-Cracking in Cement Concrete Pavements". State Highway Commission of Kansas, 1973.

Cebeci, O.Z., T. Demirel and R.A. Lohnes. "Evaluation of Hysterisis in Mercury Intrusion Porosimetry by Second-Intrusion Method". Trans. Res. Rec. No. 675, 1978, pp. 15-20.

Collins, A.R. "The Destruction of Concrete by Frost". Journal of the Institution of Civil Engineers, Vol. 23, No. 1, 1944, pp. 29-41.

Dolch, W.L. "Porosity". ASTM Special Technical Publ. 169B, 1978 pp. 646-656.

Dikeou, J.T., L.E. Kukacka, J.E. Backstrom and M. Steinberg (1969). "Polymerization Makes Tougher Concrete". ACI Journal, 66 (10): 829-839.

Domaschuk, L., G. Hermanson, and G. Ganapathy. "D-Cracking of Concrete Pavements in Manitoba", Transportation Forum Journal, Roads and Transportation Association of Canada, Sept. 1986, pp. 39-44.

Domaschuk, L., G. Ganapathy, and T. Triffo. "D-Cracking Studies of Some Manitoba Aggregates", Proceedings, R.T.A.C. Annual Conference, Sept. 1988, pp. B3-B24.

Domaschuk, L., G. Garychuk. "Pore Structure Analysis of Aggregates Susceptible to D-Cracking", Proceedings, R.T.A.C. Annual Conference, Sept. 1988, pp. B25-46.

Dunn, J.R. and P.P. Hudec. (1972). Frost and Sorption Effects in Argillaceous Rocks. Hwy. Res. Reac. No. 393: 65-78.

Dullien, F.A.L. "Photomicrographic Pore-Size Distributions Using Quantitative Stereology and Application of Results in Tertiary Petroleum Recovery". Proc. Intl. Symp. RILEM/IUPAC Prague. 1C, 1973, pp. 173-186.

Eyraud, C. et al., "Determination Carorimetrique de la Distribution des Rayons de Pores". Proc. Intl. Symp. RILEM/IUPAC Prague, 1C, 1973, pp. 81-99.

Ehrlich, R., S.K. Kennedy, S.J. Crabtree, and R.L. Cannon, "Petrographic Image Analysis: I. Analysis of Reservoir Pore Complexes". Jr. Sed. Pet. Vol. 54, No. 4. 1984. pp. 1365-1378.

Garychuk, G.W., "Study of Carbonate Aggregate Pore Systems by Image Analysis". M.Sc. Thesis, University of Manitoba, 1986.

Garychuk, G.W. "Petrographic Image Analysis (PIA) Techniques for Carbonate Rock Porosity Evaluation", Unpublished Report, University of Manitoba, 1987.

Girard, R.J.K., E.M. Meyers, G.C. Manchester, and W.L. Trimm. "D-Cracking Pavement Design and Construction Variables". Trans. Res. Rec. 853, 1982, pp. 1-9.

Hanes, F.E. and R.A. Wyman (1962). "The Application of Heavy Media Separation to Concrete Aggregate". Canadian Mining and Metallurgical Bulletin, 55(603): 489-496.

Hermanson, G.J. "D-Cracking of Concrete Pavements in Manitoba". M.Sc. Thesis, University of Manitoba, 1987.

Harvey, N. "An Investigation of Petrographic Image Analysis Techniques". B.Sc.C.E. Thesis, University of Manitoba, 1987.

Hugman, R.H.H. and M. Friedman. "Effects of Texture and Composition on Mechanical Behavior of Experimentally Deformed Carbonate Rocks". A.A.P.B. 63 (9), 1979, pp. 1478-1489.

Iowa Test Method No. 211. Method of Test for Determining the Soundness of Coarse Aggregates by Freezing and Thawing. Iowa Dept. of Transportation, Highway Division, Office of Materials, Ames, Iowa, March 1977, 3p.

Janssen, D.J. and B.J. Dempsey. "The Effect of AC Overlays on D-Cracking in PVV Pavements". Transportation Research Record, 1985.

Klieger, P. "Effect of Entrained Air on Strength and Durability of Concrete with Various Sizes of Aggregates". HRB Proceedings, pp. 1-17, 1968.

Kubie, W.L., L.E. Gast and J.C. Cowan. "Linseed Oil for the Preventative Maintenance of Concrete". HRR 254, pp. 61-67, 1967.

Kurtz, A. "Heavy Media Separation Solves Aggregate Problem". Roads and Engineering Construction, 100(2): 40-44, 1962.

Larson, T.D. "Air Entrainment and Durability Aspects of Fly Ash Concrete". ASTM Proceedings, Vol. 64, pp. 866-886, 1964.

Larson, G.H. "Effect of Substitutions of Fly Ash for Portions of Cement in Air Entrained Concrete". HRB Proceedings, Vol. 32, pp. 328-335, 1953.

Larson, T., P. Cady, M. Franzen, and J. Reed. "A Critical Review Treating Methods of Identifying Aggregates Subject to Destructive Volume Change when Frozen in Concrete and a Proposed Program of Research". Hwy. Res. Bd. Special Report 80, 1964.

Lautridou, J.P. and J.C. Ozouf. "Experimental Frost Shattering". Progress in Physical Geography, 6, 2, Centre de Geomorphologie, Centre National de La Recherche Scientifique, Caen, France, 1982, pp. 215-232.

Legg, F.E. "Freeze-Thaw Durability of Michigan Concrete Coarse Aggregates". HRB Bulletin 143, pp. 1-13.

Lin, C.H. and R.D. Walker. "Effects of Cooling Rates on the Durability of Concrete". Transportation Research Record, No. 539, 1975, pp. 8-19.

Litvan, G.G. "Phase Transititions of Absorbates: IV, Mechanism of Frost Action in Hardened Cement Paste", ACS Journal, 55(1): 38-42, 1972.

Litvan, G.G. "Pore-Structure and Frost Susceptibility of Building Materials". Proc. Intl. Symp. RILEM/IUPAC Prague 2F, 1973, pp. 17-30.

Litvan, G.G. "Adsorptive Systems at Temperatures Below the Freezing Point of the Adsorptive". Advances in Colloid and Interface Science, 9: 253-302, 1978.

Marks, V.J. and W. Dubberke. "Durability of Concrete and the Iowa Pore Index Test". Trans. Res. Rec. 853: 25-31, 1982.

Martin, R.B. "Equivalence of Porosity and Specific Surface Measures in One, Two and Three Dimensions". Proc. Int. Sym. RILEM/IUPAC Prague 1A, pp. 35-52, 1973.

McKellar, M.J. and N.C. Wardlaw. "Rock Impregnation". Dept. of Geology and Geophysics Report, University of Calgary, 1981.

Mysyk, W.K. "Petrological Studies on Carbonate Aggregate Responsible for Pavement D-Cracking in Southern Manitoba, Canada". Transportation Research Record 1110, National Research Council, Washington, D.C., pp. 10-15, 1987.

Mysyk, W.K. and R.J. Edwards. "Mineralogical Analysis and Literature Review of Limestone Aggregate Porosity/Permeability from Apron 1, Winnipeg International Airport, Winnipeg, Manitoba". Final Report to Transport Canada, Contract CCB-CON-84-24, 1985.

Paxton, J.J. "Ohio Aggregate and Concrete Testing to Determine D-Cracking Susceptibility". Trans. Res. Rec. 853: 20-24, 1982.

Powers, T.C. and R.A. Helmuth. "Theory of Volume Changes in Hardened Portland Cement Paste During Freezing". HRB Proceedings, 32: 285-297, 1953.

Powers, T.C. "A Working Hypothesis for Further Studies of Frost Resistance". ACI Proceedings, 41: 245-272, 1945.

Powers, T.C. "Basic Considerations Pertaining to Freezing and Thawing Tests". ASTM Proceedings, 55: 1132-1155, 1955.

Ravaglioli, A. and G. Vecchi. "Assessment of the Frost Resistance of Ceramic Bodies by Means of Porosity Meter Tests". Proc. Intl. Symp. RILEM/IUPAC Prague. 4F, pp. 117-128, 1973.

Rink, M. "An Image Analysis for Digital Computers and its Application to Investigations on Pore-Structures". Proc. Int. Sym. RILEM/IUPAC Prague. 6C, pp. 495-505, 1973.

Rogers, C.A., S.A. Senior and D. Boothe. "Development of an Unconfined Freeze-Thaw Test for Coarse Aggregates". Proceedings, RTAC Annual Conference, pp. B47-B68, Sept. 1988.

Stark, D. "Field and Laboratory Studies on the Effect of Subbase Type on the Development of D-Cracking". Portland Cement Association 1974. (Reprint from Highway Research Record No. 342, Highway Research Board, Washington, D.C., 1970).

Stark, D. and P. Klieger. "Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements". HRR No. 441, p. 33, 1973.

Stark, D. "Characteristics and Utilization of Coarse Aggregates Associated with D-Cracking". (RD047.01P). Portland Cement Association. Reprinted from Living with Marginal Aggregates. STP 597 ASTM Philadelphia, PA. pp. 45-58, 1976.

Thompson, S.R., M.P.J. Olsen, and B.J. Dempsey. "D-Cracking in Portland Cement Concrete". Civil Eng. Series, Trans. Eng. Series No. 29, University of Illinois, June 1980.

Traylor, M.L. "Efforts to Eliminate D-Cracking in Illinois". Concrete Analysis and Deterioration, Transport Research Record 853, Transportation Research Board, National Research Council, National Academy of Sciences, Washington, D.C. pp. 9-13, 1982.

Verbeck, G. and R. Landgren. "Influence of Physical Characteristics of Aggregates on Frost Resistance of Concrete". ASTM Proceedings, Vol. 60, pp. 1063-1079, 1960.

Verbeck, G., P. Klieger, D. Stark, and W. Teske. "Interim Report on D-Cracking of Concrete Pavements in Ohio". Ohio Department of Highways, Agreement No. 1910, March 1972.

Walker, R.D. and J.F. McLaughlin. "Effect of Heavy Media Separation on Durability of Concrete made with Indiana Gravels". HRB Bulletin 143, 1956.

Wardlaw, N.C. "Pore Geometry of Carbonate Rocks as Revealed by Pore Casts and Capillary Pressure". Bull. Am. Assoc. Pet. Geol. 60 (2), pp. 245-257, 1976.

Winslow, D.N., M.K. Lindgren, and W.L. Dolch. "Pavement D-Cracking and Coarse Aggregate Pore Structure". In Concrete Analysis and Deterioration, Transportation Research Record 853, Transportation Research Board, National Research Council, National Academy of Sciences, Washington, D.C., 1982.

BIBLIOGRAPHY

Alford, N.M. and A.A. Raham. "An Assessment of Porosity and Pore Sizes in Hardened Cement Paste." Jr. Mat. Science, 16 (1981) 3105-3114.

Aschenbrenner, B.C. et. al. "Teodorvich's Method for Determining Permeability from Pore-Space Characteristics of Carbonate Rocks." AAPG Repr. Ser. #5, 82-85 (1972).

ASTM Standards. "Resistance to Rapid Freezing and Thawing." ASTM Test C-666-80. Book of ASTM Standards, Part 14.(1983).

Axon, E.A., L.T. Murray, and R.M. Rucker. "Laboratory Freeze-Thaw Tests vs Outdoor Exposure Tests." HHR 268. Highway Research Board, Washington, D.C. 35-47 (1986).

Beaudoin, J.J. "Porosity Measurement of Some Hydrated Cementitous Systems by High Pressure Mercury Intrusion-Microstructural Limitations." Cement and Concrete Research, 9,6, 771-781 (1979).

Berry, E.E. and V.M. Malhotra. "Fly Ash in Concrete: A Critical Review." ACI Journal, 59-73. (1980).

Best, C.H. "D-Cracking in PCC Pavements, Cause and Prevention." Kansas State University Report No. KSU-EES-2422, June 1974

Brown, R.H. Pore Studies of Carbonate Aggregates by an Evaporation Technique. Ph.D. Thesis, Purdue University. (1968).

Brunauer, S., J. Skalny and I. Odler. "Complete Pore-Structure Analysis." Proc. Int. Sym. RILEM/IUPAC, Prague, Vol. 1, C, 3-26 (1973).

Bukovatz, J.E. and C.E. Crumpton. Study of D-Cracking on Portland Cement Concrete Pavements, Vol. 2, Laboratory Phase. Kansas Department of Transportation, Federal Highway Administration, March 1981.

Bukovatz, J.E., C.E. Crumpton, and H.E. Worley, Study of D-Cracking in Cement Concrete Pavements. State Highway Commission of Kansas (1973).

Bukovatz, J.E., C.E. Crumpton, and H.E. Worley, Map Concrete in Limestone-Sweetened Concrete Pavements Promotes D-Cracking. Transportation Research Record No. 65 (1977).

Cebeci, O.Z., T. Demirel and R.A. Lohnes. Evaluation of Hysterisis in Mercury Intrusion Porosimetry by Second-Intrusion Method. Trans. Res. Rec. No. 675, 15-20 (1978).

Choquette, P.W. and L.C. Pray. "Geologic Nomenclature and Classification of Porosity in Sedimentary Carbonates. AAPG Repr. Ser. #5, 154-197 (1970)

Collins, A.R. "The Destruction of Concrete by Frost." Journal of the Institution of Civil Engineers, Vol. 23, No. 1, 29-41 (1944).

Crevello, P.D., J.M. Rine, and D.E. Lanesky. "A Method for Impregnating Unconsolidated Cores and Slabs of Calcareous and Terrigenous Muds." Journal of Sedimentary Petrology. Vol. 51, No. 2, 658-660 (1981).

Dean, R.C., J.H. Haven, A.S. Rahal, and M.V. Azevedo. "Cracking in Concrete Pavement. Transportation Engineering Proceedings of ASCE, Vol. 106, No. TE2. March, 1980.

Dikeou, J.T., L.E. Kukacha, J.E. Backstrom and M. Steinberg. "Polymerization Makes Tougher Concrete". ACI Journal, 66 (10), 829-839 (1969).

Dinkle, R.E. Freezing Resistance in Mortar Related to Pore Properties of Rock Particles. MSc Thesis, University of Maryland (1966).

Dolar-Mantuani, L. "Petrographic Examination of Natural Concrete Aggregates". Hwy Res. Rec. No. 120, 7-17 (1966)

Dolch, W.D. Porosity. ASTM Special Technical Publ. 169B, 646-656. (1978).

Domaschuk, L., G. Ganapathy, and T. Triffo. "D-Cracking Studies of Some Manitoba Aggregates". Proceedings R.T.A.C. Annual Conference, B3-B24, September 1988.

Domaschuk, L. and G. Garychuk. "Pore Structure Analyses of Aggregates Susceptible to D-Cracking." Proceedings R.T.A.C. Annual Conference, B25-B46, September 1988.

Domaschuk, L., G. Hermanson and G. Ganapathy. "D-Cracking of Concrete Pavements in Manitoba". Transportation Forum Journal, Roads and Transportation Association of Canada. 39-44. Sept., 1986

Doyle, E.N. The Development and Use of Polyurethane Products. New York: McGraw-Hill Book Company, 1971. 208-211.

Dubberke, W. Factors Relating to Aggregate Durability in Portland Cement Concrete. Interim Report, Project HR-2022, Iowa Department of Transportation, Ame, Iowa, Jan. 1983.

Dubinin, M.M. "Methods of Determination of Pore-Structure, Adsorption Methods." Proc. Int. Sym. RILEM/IUPAC, Prague. 4F: 140-149 (1973).

Dullien. F.A.L. "Photomicrographic Pore-Size Distributions Using Quantitative Stereology and Application of Results in Tertiary Petroleum Recovery". Proc. Intl. Symp. RILEM/IUPAC, Prague. 1C: 173-186 (1973).

Dunn, J.R. and P.O. Hudec. "Frost and Sorption Effects in Argillaceous Rocks". Hwy. Res. Reac. No. 393. 65-78 (1972).

Ehrlich, R., S.K. Kennedy, S.J. Crabtree and R.L. Cannon. "Petrographic Image Analysis: I. Analysis of Reservoir Pore Complexes." Jr. Sed. Pet. Vol. 54, No. 4, 1365-1378. (1984).

Eyraud, C. et. al. "Determination Calorimetrique de la Distribution des Rayons de Pores." Proc. Intl. Symp. RILEM/IUPAC, Prague. 1C, 81-99 (1973).

Fagerlund, G. "Pore-Structure and Frost Resistance". Proc. Int. Symp. RILEM/IUPAC, Prague. 4F: 67-87 (1973).

Fischmeister, H.F. "Determination of Pore-Structure by Stereological Measurements." Proc. Int. Symp. RILEM/IUPAC, Prague. 6C: 439-476 (1973).

Folk, R.L. "Spectral Subdivision of Limestone Types." Classification of Carbonate Rocks: Symposium. AAPG (Memoir 1) 62-84 (1962).

Franklin, J.A. "Rock Impregnation Trials Using Monomers, Epoxide, and Unsaturated Polyester Resins." Journal of Sedimentary Petrology. Vol. 39: 1251-1253. (1969).

Gardiner, K.L. "Impregnation Technique Using Coloured Epoxy to Define Porosity in Petrographic Thin Sections." Available from Geological Survey, Box 25046, Mail Stop 904, Federal Center, Denver, Colorado, U.S.A. 80225. Undated.

Garychuk, G.W. Study of Carbonate Aggregate Pore Systems by Image Analysis. MSc Thesis, University of Manitoba (1986).

Garychuk, G.W. Petrographic Image Analysis (PIA) Techniques for Carbonate Rock Porosity Evaluation. Unpublished Report, University of Manitoba (1987).

Gilliot, J.E. "Properties of Aggregates Affecting Concrete in North America." Q. J. Eng. Geol. 13,4: 289-303 (1980).

Girard, R.J.K., E.M. Meyers, G.C. Manchester, and W.L. Trimm. "D-Cracking Pavement Design and Construction Variables." Trans. Res. Rec. 853, 1-9 (1982).

Globus, A.M. and B.M. Mogilevsky. "Thermal Transfer of Liquid in Porous Media." Hwy. Res. Board Spec. Rep. 103: 39-50 (1969).

Gribanova, E.V. and O.N. Grigorov. "Some Peculiarities of Capillary Rise of Moisture in Porous Medium." Proc. Int. Sym. RILEM/IUPAC Prague. 6C: 342-350 (1973).

Halverson, A.D. "Recycling Portland Cement Concrete Pavement." Trans. Res. Rec. 853: 14-17 (1982).

Hanes, F.E. and R.A. Wyman. "The Application of Heavy Media Separation to Concrete Aggregate." Canadian Mining and Metallurgical Bulletin, 55 (603): 489-496 (1962).

Harvey, N. An Investigation of Petrographic Image Analysis Techniques. BSc (C.E.) Thesis, University of Manitoba (1987).

Harvey, R.D. and J.C. Steinmetz. "Petrography of Carbonate Rocks by Image Analysis." 7th Proceedings: Forum on Geology of Industrial Minerals. 161-170 (1972).

Havens, J.H. The D-Cracking Phenomenon: A Case Study for Pavement Rehabilitation. Division of Research, Kentucky Bureau of Highways, Lexington, Kentucky. April 1976.

Havens, J.H. and R.C. Deen. "Possible Explanation of Concrete Pop-Outs." Trans. Res. Rec. 651: 16-24 (1977.

Hermanson, G.J. D-Cracking of Concrete Pavements in Manitoba. MSc Thesis, University of Manitoba (1987).

Hesse, D. "Pore-Structure Determination of Porous Media from Diffusion Measurement." Proc. Int. Sym. RILEM/IUPAC, Prague. 1C: 139-153 (1973).

Hilliard, J.E. "Stereology: An Experimental Viewpoint." Proceedings of the Symposium on Statistical and Probabilities Problems in Metallurgy. Applied Probability Trust: 92-111 (1972).

Hiltrop, C.L. and J. Lemish. "Relationship of Pore Size Distribution and Other Rock Properties to Serviceability of Some Carbonate Aggregates." Hwy. Res. Board Bull. No. 239: 1-23 (1960).

Honjo, S. and A.G. Fisher. Handbook of Paleontological Techniques. Bernhard Kummel and David Raup, Eds. W.H. Freeman and Co. p. 331 (1965).

Huang, J., W.C. Sherwood and N. Wakao. "Determination of Mean Pore Radii in Mineral Aggregates Using Permeability Measurement." Hwy. Res. Board Spec. Rep. No. 90: 219-233. (1966).

Hudec, P.P. "Durability of Carbonate Rocks as a Function of Their Thermal Expansion, Water Sorption and Mineralogy." ASTM Spec. Tech. Pub. 697, 497-508 (1980).

Hugman, R.H.H. and M. Friedman. "Effects of Texture and Composition on Mechanical Behavior of Experimentally Deformed Carbonate Rocks." A.A.P.B. 63 (9), 1478-1489 (1979).

Iowa Test Method No. 211. "Method of Test for Determining the Soundness of Coarse Aggregates by Freezing and Thawing." Iowa Dept. of Transportation, Highway Division, Office of Materials, Ames, Iowa. March, 1977, 3p.

Imbt, W.C. and S.P. Ellison. "Porosity in Limestone and Dolomite Reservoirs." A.P.I. Drilling and Production Practice. 364-372. (1946).

Janssen, D.J. and B.J. Dempsey. "The Effect of AC Overlays on D-Cracking in PCC Pavements." Transportation Research Record. (1985).

Kaneuji, M. Correlation Between Pore Size Distribution and Freeze Thaw Durability of Coarse Aggregate on Concrete. Purdue University, Lafayette, Ind. and Indiana State Highway Commission, Indianapolis, Indiana. Report No. FHWAIN 7815, August, 1978.

Kaneuji, M., D.N. Winslow and W.L. Dolch. "The Relationship Between an Aggregate's Pore Size Distribution and Its Freeze Thaw Durability in Concrete." Cement and Concrete Research. Vol. 10: 433-441 (1980).

Klieger, P. "Effect of Entrained Air on Strength and Durability of Concrete with Various Sizes of Aggregates." HRB Proceedings, 1-17. (1968).

Klieger, P., D. Stark and W. Teske. The Influence of Environment and Materials on D-Cracking. Portland Cement Association, Final Report. October, 1978.

Koh, Y. and E. Kamada. "The Influence of Pore-Structure of Concrete Made with Absorptive Aggregates on the Frost Durability of Concrete." Proc. Int. Sym RILEM/IUPAC, Prague. 2F: 45-62 (1973).

Koplik, J., C. Lin and M. Vermette. "Conductivity and Permeability from Microgeometry." J. Appl. Phys. 56 (11): 3127-3131 (1984).

Kubie, W.L., L.E. Gast and J.C. Cowan. "Linseed Oil for the Preventative Maintenance of Concrete." HRR 254, 61-67 (1967).

Kurtz, A. "Heavy Media Separation Solves Aggregate Problem." Roads and Engineering Construction, 100(2): 40-44 (1962).

Larson, G.H. "Effect of Substitutions of Fly Ash for Portions of Cement in Air Entrained Concrete." HRB Proceedings. Vol. 32, 328-335 (1953).

Larson, T., A. Boettcher, P. Cady, M. Franzen and J. Reed. Identification of Concrete Aggregates Exhibiting Frost Susceptibility. NCHRP Report, 15. (1965).

Larson, T. and P. Cady. Identification of Frost-Susceptible Particles in Concrete Aggregates. Hwy. Res. Board NCHRP Report 66 (1969).

Larson, T., P. Cady, M. Franzen and J. Reed. "A Critical Review Treating Methods of Identifying Aggregates Subject to Destructive Volume Change When Frozen in Concrete and a Proposed Program of Research." Hwy. Res. Bd. Special Report 80. (1964).

Larson, T.D. "Air Entrainment and Durability Aspects of Fly Ash Concrete." ASTM Proceedings, Vol. 64, 866-886 (1964).

Lankard, D.R., A.J. Walker, D.E. Niesz. Influence of Cement on Moisture Migration in Paste and Concrete as Related to the Durability of Concrete. Report No. FHWA-RE-74-54, PB 236448.

Lautridou, J.P. and J.C. Ozouf. "Experimental Frost Shattering." Progress in Physical Geography, 6, 2, Centre de Geomorphologie, Centre National de la Recherche Scientifique, Caen, France, 215-232 (1982).

Legg, F.E. "Freeze-Thaw Durability of Michigan Concrete Coarse Aggregates." HRB Bulletin. 143, 1-13. (1956).

Lin. C."Microgeometry I: Autocorrelation and Rock Microstructure." Mathematical Geology. Vol. 14, No. 4, 343-360 (1982).

Lin. C. "Microgeometry II: Testing for Homogeneity in Berra Sandstone." Mathematical Geology. Vol. 14, No. 4, 361-370 (1982).

Lin, C. "Measurement of Sphere Contacts in a Sectioning Plane." Mathematical Geology. Vol. 14, No. 5, 501-508. (1982).

Lin, C. "Shape and Texture from Serial Contours." Mathematical Geology Vol. 15. No. 5, 617-632. (1983).

Lin, C. and M.H. Cohen. "Quantitative Methods for Microgeometry Modeling." J. Appl. Phys. 53(6), 4152-4165. (1982).

Lin, C. and J. Hamanski. "Pore Geometry: A New System for Quantitative Analysis and 3-D Display." Journal of Sedimentary Petrology. 53(1), 670-672. (1983).

Lin, C.H. and R.D. Walker. "Effects of Cooling Rates on the Durability of Concrete." Transportation Research Record, No. 539, 8-19 (1975).

Lindgren, M.N. The Prediction of Freeze-Thaw Durability of Coarse Aggregate in Concrete by Mercury Intrusion Porosimeter. MSc Thesis, Dept. of Civil Engineering, Purdue University (1980).

Litvan, G.G. "Phase Transitions of Absorbates: IV: Mechanism of Frost Action in Hardened Cement Paste." ACS Journal 55(1), 38-42 (1972).

Litvan, G.G. "Pore-Structure and Frost Susceptibility of Building Materials." Proc. Int. Sym. RILEM/IUPAC, Prague. 2F: 17-30 (1973).

Litvan, G.G. "Absorption Systems at Temperatures Below the Freezing Point of the Absorptive." CNRC Rep. 183. 4:283-302 (1978).

Litvan, G.G. "Adsorptive Systems at Temperatures Below the Freezing Point of the Adsorptive" Advances in Colloid and Interface Science, 9:253-302 (1978).

Litvan, G.G. "Freeze-Thaw Durability of Porous Building Materials." ASTM Spec. Tech. Publ. 691: 455-463 (1980).

Litvan, G.G. Frost Action in Porous Systems. Div. of Bldg. Res. Paper No. 1176 (1981).

Longman, M.W. and P.A. Mench. "Diagenesis of Cretaceous Limestones in the Edwards Aquifer System South-Central Texas: A Scanning Electron Microscope Theory." Sedimentary Geology 21: 241-276 (1978).

Longman, M.W. "Carbonate Diagenetic Textures From Nearsurface Diagenetic Environments." AAPG Bull. 16, 4: 461-487 (1980).

MacInnis, C. and J.J. Beaudoin. "Pore-structure and Frost Durability." Proc. Int. Sym. RILEM/IUPAC Prague. 2F: 3-15 (1973).

Marks, V.J. and W. Dubberke. "Durability of Concrete and the Iowa Pore Index Test." Trans. Res. Rec. 853: 25-31 (1982).

Martin, R.B. "Equivalence of Porosity and Specific Surface Measures in One, Two and Three Dimensions." Proc. Int. Sym. RILEM/IUPAC Prague. 1A: 35-52 (1973).

McKellar, M.J. and N.C. Wardlaw. Rock Impregnation. Dept. of Geology and Geophysics Report, University of Calgary (1981).

McLellan, A.F. Report on the Freeze Thaw Durability Tests with Limestone Aggregates. Manitoba Department of Highways, Material Research Branch, June, 1979.

Meininger, R.C. Influence of Aggregate Pore Properties on the Resistance of Concrete to Freezing Damage. MSc Thesis, University of Maryland (1964).

Meyer, L.P.H. Determination of Air Content of Hardened Concrete Using Image Analysis. MSc Thesis, University of Windsor (1983).

Ministr, Z. "Stereology and Mercury Porosimetry." Proc. Int. Sym. RILEM/IUPAC Prague. 1C: 155-171 (1973).

Minoura, N. and C.D. Conley. "Technique for Impregnating Porous Rock Samples with Low-Viscosity Epoxy Resin." Journal of Sedimentary Petrology. Vol. 41, No. 3: 858-861 (1971).

Missouri State Highway Department. Investigation of D-Cracking in PCC Pavements Phase 3. Missouri Cooperative Highway Research Program, Report No. 71-3, Jefferson City (1977).

Murphy, C.P., P. Bullock and R.H. Turner. "The Measurement and Characterization of Voids in Soil Thin Sections by Image Analysis. Part I: Principles and Techniques." Jr. Soil Science, 28: 498:508 (1977).

Myers, J.D. and W. Dubberke. Iowa Pore Index Test. Iowa Department of Transportation, January, 1980.

Mysyk, W.K. "Petrological Studies on Carbonate Aggregate Responsible for Pavement D-Cracking in Southern Manitoba, Canada." Transportation Research Record 1110, National Research Council, Washington, D.C., 10-15. 1987.

Mysyk, W.K. and R.J. Edwards. Mineralogical Analysis and Literature Review of Limestone Aggregate Porosity/Permeability from Apron 1, Winnipeg International Airport, Winnipeg, Manitoba. Final Report to Transport Canada, Contract CCB-CON-84-24 (1985).

Nazare, S. and G. Ondrocek. "Characteristics of Microstructure and Its Effects on the Properties of Porous Materials." Proc. Int. Sym. RILEM/IUPAC Prague. 6C: 477-495. (1973).

Nuss, W.F. and R.L. Whiting. "Technique for Reproducing Rock Pore Space." American Association of Petroleum Geologists Bull. Vol. 31: 2044-2049 (1947).

Nutting, P.G. "Some Physical Problems in Oil Recovery." Oil and Gas Journal. Vol. 28, No. 27, 44-45 (1929).

Olsen, M.P.J., D.J. Hanssen and B.J. Dempsey. Final Report: D-Cracking in Portland Cement Concrete Pavements. Trans. Eng. Ser. No. 37, University of Illinois (1983).

Olsen, M.P.J. "Mathematical Modeling of the Freezing Processing in Concrete and Aggregates." Cem. Concrete Res. 14, 1:113-12[illegible]. (1984).

Ondracek, G. "Mean Stereometric Factors and Properties of Porous Materials." Proc. Int. Sym. RILEM/IUPAC, Prague (1973).

Paxton, J.T. Ohio Aggregate and Concrete Testing to Determine D-Cracking Susceptibility in Concrete Analysis and Deterioration. Transportation Research Record 853, Transportation Research Board, National Research Council, National Academy of Sciences, Washington, D.C. (1982).

Paxton, J.T. and W.R. Feltz. Development of Laboratory Methods for Determining D-Cracking Susceptibility of Ohio Gravel and Limestone Coarse Aggregate in Concrete Pavements. Ohio Dept. of Transportation, Final Report, October 1979.

Petruk, W. "Image Analysis in Process Mineralogy." Process Mineralogy: II: Applications in Metallurgy, Ceramics and Geology. New York: Am Inst. Min., Metall. and Pet. 39-53 (1982).

Pimenov. V.V., B. Vladimir and A. Angelina. "Structure of Pores and Ice Formation Processes in Concrete." Proc. Int. Sym. RILEM/IUPAC Prague. 4F: 89-98. (1973).

Pittman, E.D. and R.W. Duschatko. "Use of Pore Casts and Scanning Electron Microscope to Study Pore Geometry." Journal of Sedimentary Petrology. Vol. 40, No. 4: 1153-1157 (1970).

Pittman, E.D. and J.B. Thomas. "Some Applications of SEM to the Study of Reservoir Rock." Jnl. Pet. Tech. 31(11): 1375-1380. (1979).

Podvalmy, A.M. and A.M. Prozenko. "An Investigation of the Permeability of Porous Bodies on a Mathematical Model." Proc. Int. Sym. RILEM/IUPAC Prague. 3A: 13-31 (1973).

Portland Cement Association. Design and Control of Concrete Mixtures. Eng. Bull. (1979).

Powers, T.C."Basic Considerations Pertaining to Freezing and Thawing Tests." ASTM Proceedings, Vol. 55, 1132-1155 (1955).

Powers, T.C. "A Working Hypothesis for Further Studies of Frost Resistance." ACI Proceedings, 41: 245-272 (1945).

Powers, T.C. and R.A. Helmuth. "Theory of Volume Changes in Hardened Portland Cement Paste During Freezing." HRB Proceedings, 32, 285-297 (1953).

Racic, D. An Investigation of the Effect of Super Plasticizers on the Durability of Concretes Containing Various Coarse Aggregates. MSc Thesis, University of Maryland (1962).

Ravaglioli, A. and Vecch, G. "Assessment of the Frost Resistance of Ceramic Bodies by Means of Porosity Meter Tests." Proc. Int. Sym. RILEM/IUPAC Prague. 4F: 117-128 (1973).

Reilly, R.J. The Effect of Moisture Content and Distribution on the Durability of Concretes Containing Various Coarse Aggregates. MSc Thesis, University of Maryland (1962).

Renton, J.J. A Pressure Chamber for the Impregnation of Porous Rock Specimens. Circular 5, State of West Virginia Geological and Economic Survey. January, 1967.

Rink, M. "An Image Analysis for Digital Computers and Its Application to Investigations on Pore Structure. Proc. Int. Sym. RILEM/IUPAC Prague. 6C: 495-505. (1973).

Rink, M. and J.R. Schopper. "Pore-Structure and Physical Properties of Porous Sedimentary Rocks." Pure Appl. Geophys. 114: 273-284, (1976).

Rink, M. and J.R. Schopper. "On the Application of Image Analysis to Formation Evaluation." Log Anal. Vol. 19, No. 1, 12-22 (1978).

Robinson, R.B. "Classification of Reservoir Rocks by Surface Texture." AAPG Report Ser. #5: 95-107 (1972).

Rogers, C.A., S.A. Senior and D. Boothe. "Development of an Unconfined Freeze-Thaw Test for Coarse Aggregates." Proceedings RTAC Annual Conference, B47-B68, Sept. 1988.

Royse, C.F. Jr., J.S. Waddel, and L.E. Peterson. "X-Ray Determination of Calcite-Dolomite: An Evaluation." Jnl. Sed. Pet. Vol. 41, No. 2, 483-488 (1971).

Scheidegger, A.E. "Models and Geometry of Pore-Structure." Proc. Int. Sym. RILEM/IUPAC Prague. 3A: 3-12 (1973).

Schopper, J.R. "A Statistical Network Model and Theory of Porous Media." Proc. Int. Sym. RILEM/IUPAC Prague, 1A: 53-71 (1973).

Shakoor, A., T.R. West and C.F. Scholer. "Physical Characteristics of Some Indiana Argillaceous Carbonates Regarding Their Freeze-Thaw Resistance in Concrete." Bull. Assoc. Eng. Geologists. Vol. 19, 371-384 (1982).

Sing, K.S.W. "Formation of Pore-Structure." Proc. Int. Sym. RILEM/IUPAC Prague. 3B: 5-35 (1973).

Stark, D. Field and Laboratory Studies on the Effect of Subbase Type on the Development of D-Cracking. Portland Cement Association (Reprint from Highway Research Record No. 342, Highway Research Board, Washington, D.C. 1970) (1974).

Stark, D. Characteristics and Utilization of Coarse Aggregates Associated with D-Cracking. RD047.01P, Portland Cement Assoc. Reprinted from Living with Marginal Aggregates, STP 597, ASTM Philadelphia, Pa. 45-58 (1976).

Stark, D. and P. Klieger. "Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements." HRR No. 441, 33 (1973).

Straley, C. "A Brominated Epoxy Embedding System." Powder Technology No. 35, 259-261 (1983).

Talash, A.W. and P.B. Crawford. "Rock Properties Computed from Random Pore-Size Distribution." Jnl. Sed. Pet. 35(4): 917-921 (1965).

Taylor, R.P. and N.C. Wardlaw. "Increased Precision of Porosity Measurements Using a Modified Ruska Universal Porometer." Jr. Can. Pet. Tech. 14: 33-37 (1975).

Terrell, A.C. "Recent Advances in Automatic Image Analysis." Proc. Int. Sym. RILEM/IUPAC Prague. 6C: 506-515 (1973).

Thompson, S.R., M.P.J. Olsen and B.J. Dempsey. D-Cracking in Portland Cement Concrete. Civil Eng. Series, Trans. Eng. Series No. 29, University of Illinois, June, 1980.

Timur, A., W.B. Hempkins and R.M. Weinbradt. "Scanning Electron Microscope Study of Pore Systems in Rocks." Jnl. Geophys. Res. 76, 20: 4932-4948 (1971).

Traylor, M.L. "Efforts to Eliminate D-Cracking in Illinois." Concrete Analysis and Deterioration, Transportation Research Council, National Academy of Sciences, Washington, D.C. 9-13 (1982).

Valenta, O. "Main Properties of the Pore Systems of Non-Metallic Structural Materials, Their Significance and Experimental Determination." Proc. Int. Sym. RILEM/IUPAC Prague. 2E: 75-101 (1973).

Venecanin, S.D. "Influence of Temperature on Deterioration of Concrete in the Middle East." Concrete, 11, 8, 31-32 (1977).

Verbeck, G. and R. Landgren. "Influence of Physical Characteristics of Aggregates on Frost Resistance of Concrete." Proceedings of the American Society for Testing and Materials, Vol. 60 (1960).

Verbeck, G., P. Klieger, D. Stark, and W. Teske. Interim Report on D-Cracking of Concrete Pavements in Ohio. Ohio Department of Highways, Agreement No. 1910, March 1972.

Waldo, A.W. and S.T. Yuster. "Method of Impregnating Porous Materials to Facilitate Pore Studies." American Association of Petroleum Geologists Bulletin. Vol. 21, 259-267 (1937).

Walker, R.D. Identification of Aggregates Causing Poor Concrete Performance When Frozen. National Cooperative Highway Research Program Report 12 (1965).

Walker, R.D. Identification of Coarse Aggregates that Undergo Destructive Volume Changes when Frozen in Concrete. Hwy. Res. Rec. 120, 1-6 (1966).

Walker, R.D. and T. Hsich. Relationship between Aggregate Pore Characteristics and Durability of Concrete Exposed to Freezing and Thawing. Hwy. Res. Rec. 226, 41-49 (1968).

Walker, R.D. and J.F. McLaughlin. Effect of Heavy Media Separation on Durability of Concrete made with Indiana Gravels." HRB Bulletin 143 (1956).

Wardlaw, N.C. and R.P. Taylor. "Mercury Capillary Pressure Curves and the Interpretation of Pore Structure and Capillary Behaviour in Reservoir Rocks." Bull. Cdn. Pet. Geol. 24, 2 225-262 (1976).

Wardlaw, N.C. "Pore Geometry of Carbonate Rocks as Revealed by Pore Casts and Capillary Pressure." Bull. Am. Assoc. Pet. Geol. 60, 2, 245-257 (1976).

Wardlaw, N.C. Pore Systems in Carbonate Rocks and Their Influence on Hydrocarbon Recovery Efficiency: Short Course on Carbonate Porosity. AAPG Continuing Education Course Note Series #11 (1979).

Wardlaw, N.C. "Mercury Porosimetry and the Interpretation of Pore Geometry in Sedimentary Rocks and Artificial Models." Powder Technology. 29: 127-143 (1981).

Williams, F.M., A. Trenfy, J.T. Paxton, and H.D. Davis. Developments of Laboratory Methods for Determining D-Cracking Susceptibility of Ohio Gravel and Limestone Coarse Aggregates in Concrete Pavements. Ohio Dept. of Transportation, Final Report, October, 1974.

Wills, M.H. Jr., H.A. Lepper, R.D. Gaynore, and S. Walker. Volume Change as a Measure of Freezing and Thawing Resistance of Concrete made with Different Aggregate. Proceedings of the American Society for Testing and Materials (1963).

Winsauer, W.O., H.M. Shearin Jr., P.H. Masson and M. Williams. "Resistivity of Brine-saturated Sands in Relation to Pore Geometry." American Association of Petroleum Geologists Bull. Vol. 36, 253-277 (1952).

Winslow, D.N., M.K. Lindgren, and W.L. Dolch. Pavement D-Cracking and Coarse Aggregate Pore Structure, in Concrete Analysis and Deterioration, Transportation Research Record 853, Transportation Research Board, National Research Council, National Academy of Sciences, Washington, D.C. (1982).

Zagar, L. "Methods of Determination of Pore-Structure Permeability and Diffusion." Proc. Int. Sym. RILEM/IUPAC Prague. 6C: 249-277. (1973).

APPENDIX A

PETROGRAPHIC EVALUATION OF CARBONATE ROCKS TO DETERMINE D-CRACKING SUSCEPTIBILITY

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INTRODUCTION

The petrographic study consisted of a description of over 200 SEM photographs from 28 carbonate aggregate particles. Grain morphology, texture and porosity types were the features described in the photographs. Rock types were named from the x-ray diffraction data as follows:

% Dolomite	% Calcite	Rock Name
100	0	Dolomite
90	10	Calcitic Dolomite
50	50	
10	90	Dolomitic Limestone
0	100	Limestone

CONTROL SAMPLES

FT4 103 Limestone

The sample consists of some larger anhedral calcite grains in a micrite matrix. Porosity is very low (approximately 1%) which should compensate for the somewhat narrow pore size range that could cause D-cracking problems. Some fossil remnants have been completely replaced by calcite. No clay or sulfate minerals occur.

FT4 106 Dolomite

Anhedral to subhedral dolomite grains are the dominant feature, resulting in a xenotopic, equigranular mosaic texture. Remnants of finer-grained micrite coating on some dolomite grains are probably relic features from the original texture. Traces of platy clay minerals are visible at higher magnification. Intergranular porosity is the only type present; overall porosity may be approximately 2 to 3%. A few small solution pits occur in some dolomite grains.

FT4 108 Limestone

Anhedral calcite grains in a micrite matrix is the main feature. One large calcite grain displays numerous solution pits. Intergranular porosity is approximately 1 to 2%. No clay minerals are visible.

FT4 109 Limestone

The sample consists of anhedral calcite grains in a micrite matrix, resulting in a bimodal grain size distribution. The step-like features visible on some of the larger calcite grains are due to the intersection of cleavage planes and fractures. Intergranular porosity is estimated to be approximately 1 to 2%. No clay minerals are visible. A few solution pits are visible in some larger calcite grains. Although dolomite is present (up to 7% as determined by x-ray diffraction) it is not possible to distinguish the dolomite from calcite due to the anhedral nature of the grains.

FT5 18C Limestone

The sample has a bimodal size distribution consisting largely of anhedral to subhedral calcite in a micrite matrix. Some remnant fossil fragments have been replaced by calcite. Porosity is very low (1 to 2%) and is intergranular. Solution pits may be observed on some larger calcite grains, but are a very minor feature. No clay minerals are visible.

The very low intergranular porosity and absence of clay minerals means the limestone should be a relatively durable aggregate.

FT5 21B Dolomite

The sample consists mainly of subhedral to (lesser amounts of) euhedral dolomite. A very small amount of micrite occurs as interstitial material, but does not register on the x-ray diffractogram. Solution pits are a minor feature of the dolomite. The very low intergranular porosity (1-2%) and absence of clay minerals means the dolomite should be a durable aggregate. Pore sizes are large enough to prevent D-cracking deterioration.

DURABLE TEST SAMPLES

S1-11 Dolomite

Larger subhedral to (rarely) euhedral dolomite grains sit in a dolomitic micrite matrix. The intergranular pore size range may be large enough to prevent freeze-thaw problems. No clay or sulfate minerals are visible. Solution pits are a very minor feature of some larger dolomite grains.

S1-12 Dolomite

Larger anhedral to subhedral dolomite grains are embedded in a dolomitic micrite matrix. Intergranular porosity is the only type present. The pore size range may be large enough to prevent freeze-thaw deterioration as the dolomite grains are much larger than the micrite. No clay or sulfate minerals are visible.

S1-13 Dolomite

Large euhedral dolomite grains are the dominant feature. A minor amount of interstitial dolomitic micrite occurs. Platy clay minerals occur in trace amounts on the surface of a few euhedral dolomite grains. The clays show no sign of promoting deterioration. Solution pits are a very minor feature of some dolomite grains. Intergranular porosity (5 to 6% est.) is the only type present. Pore size range is relatively large due to the variable size of the dolomite crystals.

S1-14 Limestone

The rock is composed of roughly equal proportions of larger anhedral calcite grains in a micrite matrix. Although dolomite comprises 6% of the sample (from XRD data), the dolomite cannot be distinguished from calcite in the SEM photographs. Traces of clays and possibly a sulfate mineral (anhydrite?) are randomly scattered in the sample; the clays and sulfates are not present in sufficient quantity to cause problems. Intergranular porosity is estimated to be 2 to 3% and is the only type present. Pore size distribution has a relatively wide range due to the variability of the grain size.

S1-15 Dolomite

The sample is composed of large subhedral to euhedral dolomite crystals of relatively uniform size. A few small solution pits occur in some dolomite grains. A few dolomite grains appear to have been subjected to severe dissolution. Traces of a sulfate mineral occur where the dissolution of the dolomite is most severe. No clay minerals are present. Intergranular porosity is approximately 3 to 4%. Pore size range is relatively narrow due to the uniform grain size.

Sl-16 Calcitic Dolomite

The sample consists of larger subhedral to euhedral dolomite grains with interstitial micrite and somewhat larger anhedral calcite grains. Solution pits are a minor feature of some dolomite grains. No clay or sulfate minerals are noted. The variable size and shape of the dolomite, calcite and micrite likely provide a wide enough pore size range to prevent freeze-thaw deterioration. Intergranular porosity is estimated to be 2 to 3%. Although the composition of the sample is in the range that could be susceptible to the alkali-carbonate rock reaction, the lack of clay minerals rules out this possibility.

Sl-17 Dolomite

The sample consists of equigranular mosaic, anhedral to subhedral dolomite grains. Solution pits are a very minor feature of some dolomite grains. No clay or sulfate minerals occur. Intergranular porosity is estimated to be 1 to 2%. The relatively uniform size of the dolomite grains may be offset by the irregular shapes which may produce a wide enough pore size range to prevent freeze-thaw deterioration.

Sl-18 Dolomite

The rock is composed of equigranular mosaic, anhedral to subhedral dolomite grains. Solution pits occur in a few dolomite grains. No clay or sulfate minerals occur. Intergranular porosity is estimated to be 2 to 3%. Grain size and shape appears to be variable enough to produce a relatively wide pore size distribution that would prevent freeze-thaw deterioration.

Sl-19 Silicified Dolomite

The sample consists largely of fine-grained, anhedral quartz which has replaced dolomite and calcite. Euhedral to subhedral dolomite grains are still prominent, both from the x-ray diffraction data and the SEM photos. The dolomite grains are coated with small anhedral quartz(?) and even finer platy clay minerals. Calcite is present in trace amounts from x-ray diffraction data. Intergranular porosity is low (estimated at 2%) and the pore size range is large due to the great size difference between the dolomite grains and the small anhedral quartz. Several photos of the cement paste show it to be quite porous, but likely not very permeable.

Sl-20 Dolomitic Limestone

The SEM photos show mainly larger euhedral dolomite grains, with some anhedral calcite grains and a very small amount of micrite. Solution pits are common in the dolomite crystals; some dolomite grains show signs of more intense solution processes (rough surface area where about 10 to 20% of a formerly euhedral crystal has been dissolved away). No clay or sulfate minerals are visible. Porosity types present are intercrystal and intergranular. Pore size range is relatively wide and the pore sizes are generally large due to the rather coarse-grained nature of the dolomite.

NOTE: The SEM photos show mainly dolomite and may not be representative of the sample, as x-ray diffraction data reveals the rock to be only 23% dolomite.

Sl-21 Dolomite

The rock consists of anhedral to subhedral (rarely euhedral) dolomite grains with a relatively wide size range. Solution pits are a very minor feature that occur in a few dolomite grains. No clay minerals are visible, but trace amounts of a whitish, rounded, anhedral sulfate(?) mineral occur as coatings on a few dolomite grains. Intergranular porosity is estimated to be 4 to 5%. The variability of grain size and shape appears to result in a wide pore size range that would prevent freeze-thaw deterioration.

S1-20 Dolomitic Limestone

The SEM photos show mainly larger euhedral dolomite grains, with some anhedral calcite grains and a very small amount of micrite. Solution pits are common in the dolomite crystals; some dolomite grains show signs of more intense solution processes (rough surface area where about 10 to 20% of a formerly euhedral crystal has been dissolved away). No clay or sulfate minerals are visible. Porosity types present are intercrystal and intergranular. Pore size range is relatively wide and the pore sizes are generally large due to the rather coarse-grained nature of the dolomite.

NOTE: The SEM photos show mainly dolomite and may not be representative of the sample, as x-ray diffraction data reveals the rock to be only 23% dolomite.

S1-21 Dolomite

The rock consists of anhedral to subhedral (rarely euhedral) dolomite grains with a relatively wide size range. Solution pits are a very minor feature that occur in a few dolomite grains. No clay minerals are visible, but trace amounts of a whitish, rounded, anhedral sulfate(?) mineral occur as coatings on a few dolomite grains. Intergranular porosity is estimated to be 4 to 5%. The variability of grain size and shape appears to result in a wide pore size range that would prevent freeze-thaw deterioration.

NON-DURABLE TEST SAMPLES

BP2-1 Dolomite

Euhedral to subhedral dolomite crystals are overgrown with micrite to varying degrees. The micrite overgrowth is likely a later stage diagenetic feature deposited from shallow (phreatic) groundwater circulation. Intergranular porosity is estimated to be approximately 4 to 5%. No clay minerals are visible. Solution pits are a common feature on the surface of the dolomite grains.

BP2-2 Silicified Limestone

The sample consists almost entirely of very fine-grained, rounded, anhedral quartz. A minor amount of calcite and dolomite is present (from x-ray diffraction data) although the carbonates cannot be distinguished in the SEM photographs due to the anhedral grain shape. No clay or sulfate minerals are present as indicated by both SEM photos and x-ray diffraction data. Intergranular porosity is approximately 1%; the extremely uniform grain size and shape results in a narrow pore size range that may be responsible for freeze-thaw deterioration.

The sample is a strongly silicified limestone that has been noted as a rock type that is strongly prone to D-cracking deterioration in other studies.

BP2-3 Dolomite

Euhedral to subhedral dolomite grains are the most prominent feature with interstitial micrite. Intergranular porosity is estimated to be 3 to 4%. Pore size range is relatively narrow. A trace amount of platy clay minerals is visible at higher magnification.

BP2-4 Dolomite

Anhedral to subhedral dolomite grains are the dominant feature. Interstitial micrite or other minerals are present in trace amounts only. Intergranular porosity is estimated to be about 2 to 3%. The uniform grain size and shape result in a narrow pore size range that is a negative factor in freeze-thaw durability. Clay minerals are present as a trace amount only.

BP2-5 Dolomite

The sample consists of anhedral to subhedral dolomite with apparently no interstitial micrite. Intergranular porosity is estimated at 2 to 3%. Pore size range is narrow due to the uniform grain size and shape. A trace amount of platy clay minerals occur.

BP2-6 Calcitic Dolomite

Euhedral equigranular dolomite grains are the most prominent feature, with interstitial micrite. Solution pits are common in the dolomite grains. No clay minerals are visible. Intergranular porosity would normally be significant for a rock composed of euhedral dolomite, but the large amount of interstitial micrite greatly reduces the porosity which may be 2 to 3%. Pore size range should be relatively large given the large size difference between the dolomite and micrite and the fact that dolomite is only slightly more abundant than the micrite. The composition of the sample could make it susceptible to an alkali-carbonate rock reaction, but the absence of clay minerals rules out this possibility.

BP2-7 Dolomite

Anhedral to (rarely) euhedral dolomite grains sit in a micrite matrix. Solution pits are common in the dolomite grains. Overall porosity would be low (2 to 3%) and the pore size range of the intergranular porosity would be small due to the relatively small difference in size between the dolomite grains and the micrite matrix. No clay minerals are visible.

BP2-8 Dolomite

Anhedral to (rarely) subhedral dolomite grains form an equigranular mosaic texture with a low overall porosity (approximately 3 to 4%). The pore size range is very narrow due to the uniform size and shape of the dolomite grains. The rock is a very clean carbonate with no clay minerals visible.

BP2-9 Silicified Dolomite

The sample consists almost entirely of very fine-grained, round, anhedral quartz. X-ray diffraction data reveals dolomite is a minor constituent and calcite occurs as a trace mineral. No clay or sulfate minerals occur in the x-ray diffractogram, although a trace of platy clay minerals is visible in the SEM photographs. Intergranular and vuggy porosity are estimated to be 2 to 3%; grain size and shape are uniform enough to result in a relatively narrow pore size range that is detrimental to feeze-thaw durability.

S1-9 Dolomitic Limestone

The very fine-grained micrite matrix is the dominant mineralogical feature. Euhedral dolomite cyrstals and anhedral calcite grains are scattered randomly in the micrite. Solution pits are common in the dolomite crystals. Intergranular porosity is low (2 to 3%) and the pore size range would be narrow as the dolomite and calcite grains are not that much larger than the micrite matrix. Clay minerals are not visible.

Sl-10 Dolomite

The relatively uniform size and shape of the anhedral to subhedral dolomite grains results in a narrow intergranular pore size range. Porosity is estimated to be 2 to 3%. Part of the sample is a very clean dolomite with no clay or sulfate minerals, but part of it contains scattered anhedral sulfate grains (anhydrite?) which are likely a late stage diagenetic feature.

APPENDIX B

D-CRACKING SURVEY

CITY OF WINNIPEG DISTRICT 3, REPRESENTING OVER 3300 READINGS

1956	
%	ADS
82.0	1.95
72.0	1.64
77.0	1.80

1957	
%	ADS
82.0	1.89

1958	
%	ADS
68.0	1.44
54.0	0.89
61.0	1.17

1960	
%	ADS
56.0	1.0
60.0	1.9
100.0	2.2
82.0	1.6
75.0	1.7

1963	
%	ADS
88.0	1.9
78.0	1.7
62.0	1.2
86.0	1.9
79.0	1.7

1966	
%	ADS
59.0	1.2
89.0	1.7
74.0	1.5

1967	
%	ADS
79.0	1.3
70.0	1.4
75.0	1.8
75.0	1.5

1969	
%	ADS
81.0	1.6

1971	
%	ADS
62.0	1.12

1972	
%	ADS
34.0	0.74

1974	
%	ADS
63.0	1.20
84.0	2.16
84.0	1.94
80.0	1.38
91.0	2.02
61.0	1.01
77.0	1.62

1975	
%	ADS
18.0%	0.28

1976	
%	ADS
48.0	0.76
56.0	0.78
49.0	0.66
51.0	0.73

1977	
%	ADS
26.0	0.41
44.0	0.66
48.0	0.71
58.0	0.81
44.0	0.65

1978	
%	ADS
55.0	0.65
28.0	0.46
44.0	0.71
56.0	0.84
52.0	0.68
21.0	0.26
43.0	0.60

1979	
%	ADS
26.0	0.51

LEGEND

%: Percent joints affected
ADS: Average distress severity

D-CRACKING SURVEY

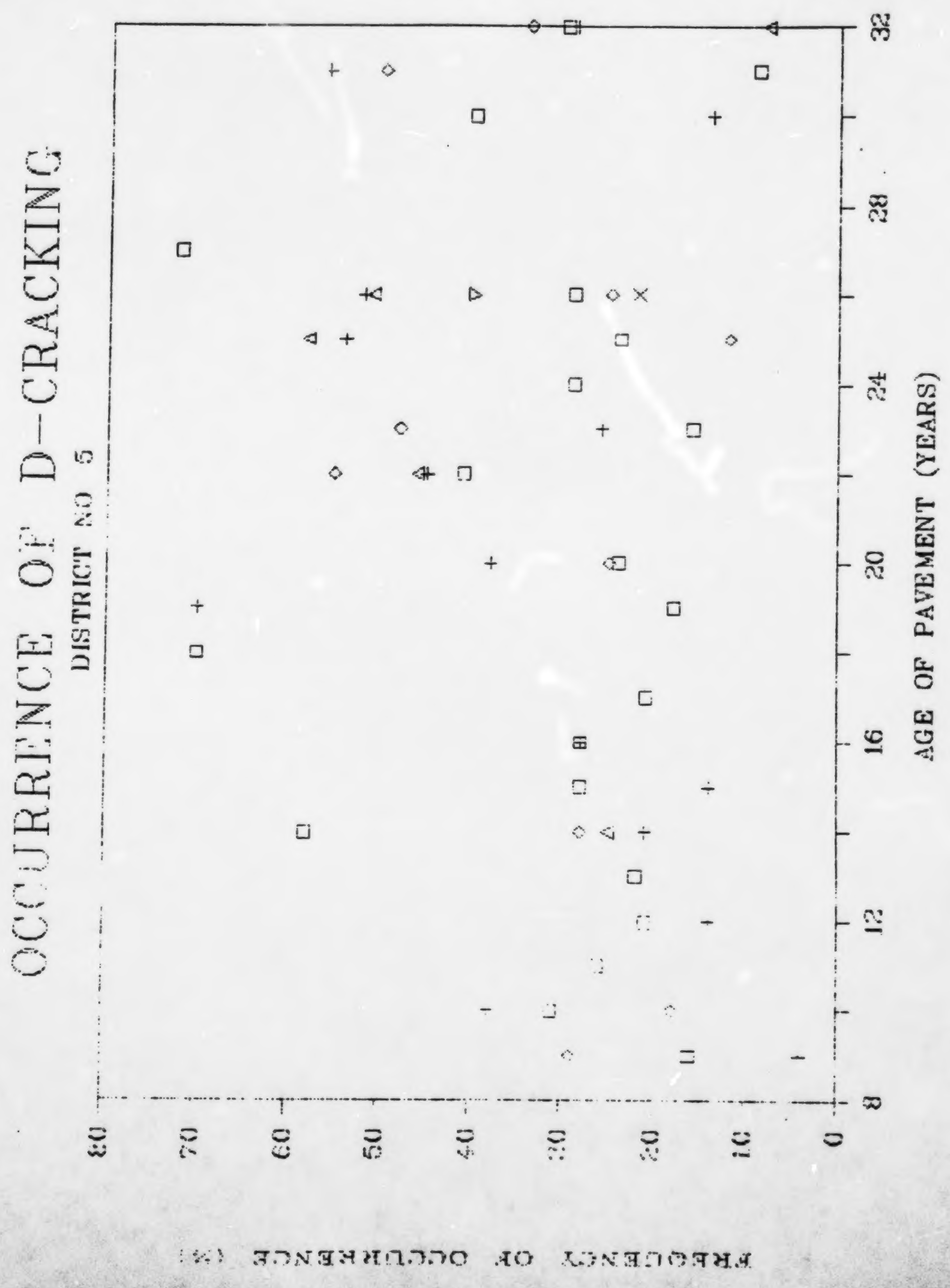
CITY OF WINNIPEG DISTRICT 2, REPRESENTING OVER 3900 READINGS

1957		1958		1960		1961		1963		1964		1965	
%	ADS	%	ADS	%	ADS	%	ADS	%	ADS	%	ADS	%	ADS
68.0	0.98	88.0	1.32	78.0	1.22	45.0	0.54	74.0	1.14	44.0	0.68	54.0	0.50
45.0	0.88	66.0	0.95			72.0	1.08	62.0	1.05	38.0	0.65		
78.0	1.28	70.0	1.10					94.0	1.53	54.0	0.75		
64.0	0.96	68.0	1.16							80.0	1.62		
72.0	1.28	85.0	1.61							59.0	0.96		
										54.0	0.65		
										74.0	1.55		
										66.0	1.50		
										61.0	0.65		
										42.0	0.72		
										54.0	0.80		
										28.0	0.38		
										66.0	1.02		
										60.0	0.95		
65.4	1.08	75.4	1.23			58.5	0.81	76.7	1.24	55.7	0.92		

1966		1968		1970		1971		1973		1974		1975	
%	ADS	%	ADS	%	ADS	%	ADS	%	ADS	%	ADS	%	ADS
48.0	0.58	72.0	1.01	70.0	1.07	44.0	0.55	66.0	0.92	54.0	0.87	45.0	0.56
48.0	0.76			76.0	1.25			52.0	0.64			38.0	0.59
												62.0	0.92
												64.0	1.51
48.0	0.67			73.0	1.16			59.0	0.78			52.3	0.90

1976		1978		1979	
%	ADS	%	ADS	%	ADS
51.0	0.86	25.0	0.25	34.0	0.36
		80.0	1.52	39.0	0.56
		52.5	0.89	36.5	0.46

LEGEND
%: Percent joints affected
ADS: Average distress severity

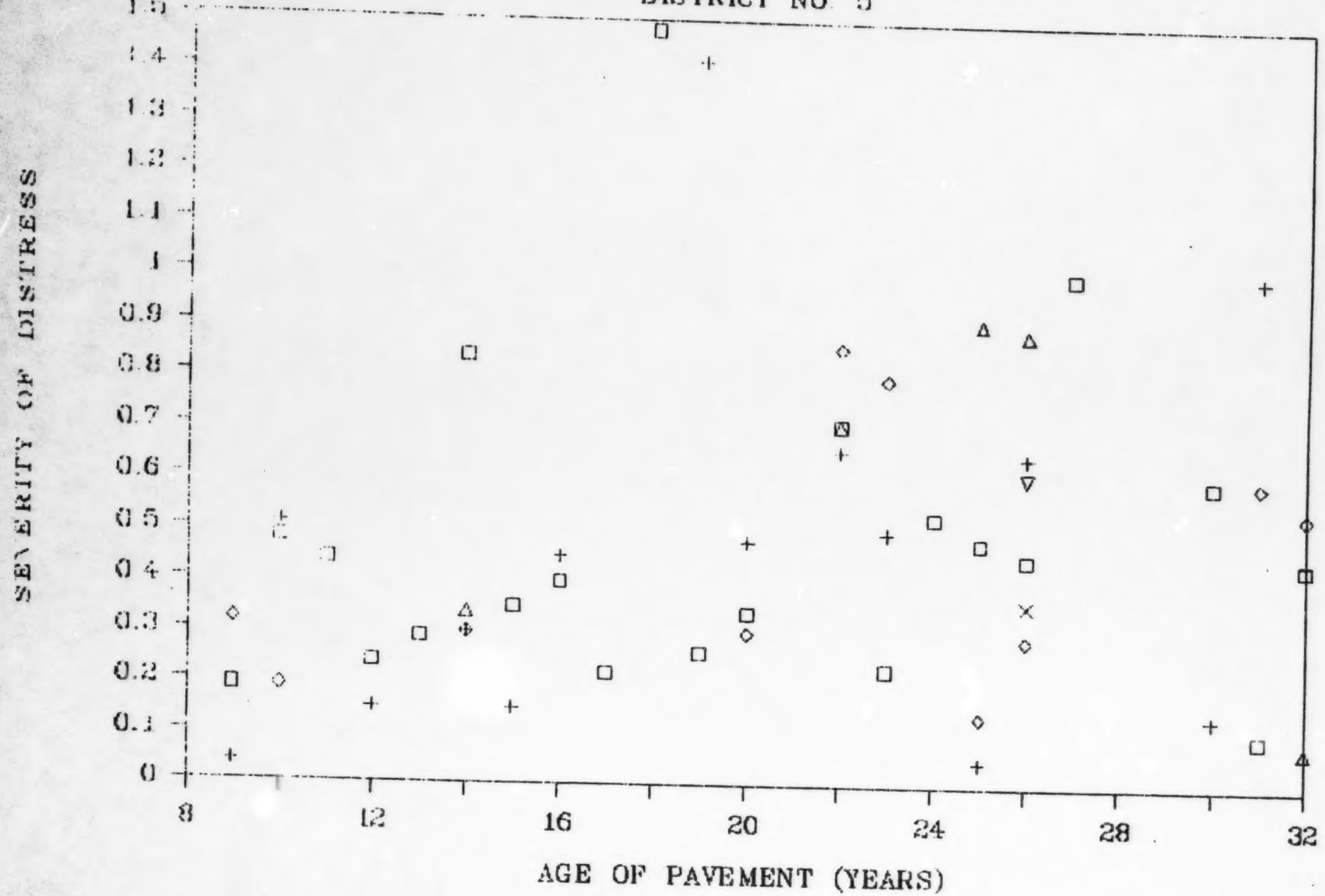


OCCURRENCE OF D-CRACKING

DISTRICT NO 5

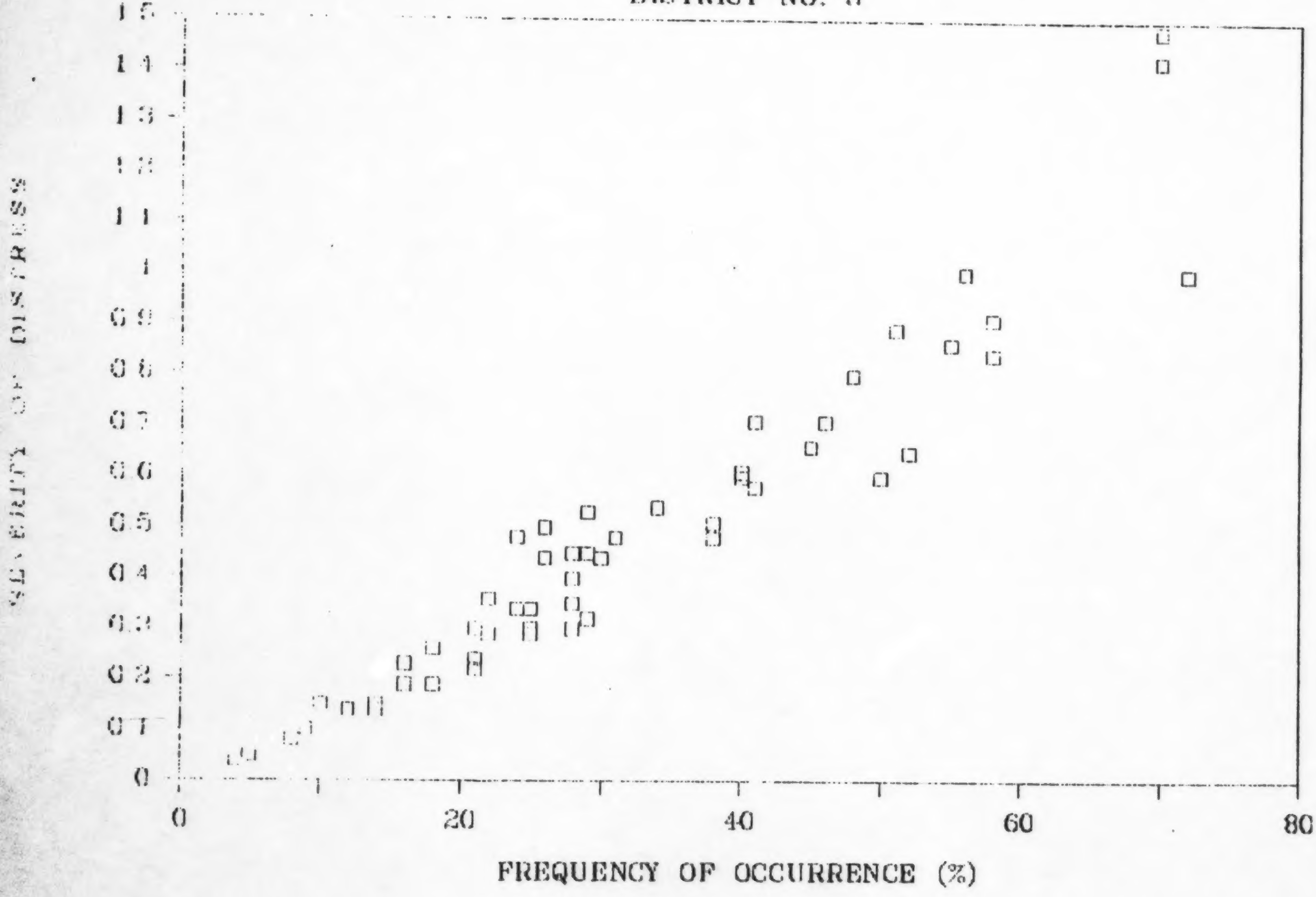
SEVERITY OF D-CRACKING

DISTRICT NO. 5



FREQUENCY OF OCCURRENCE vs SEVERITY

DISTRICT NO. 5



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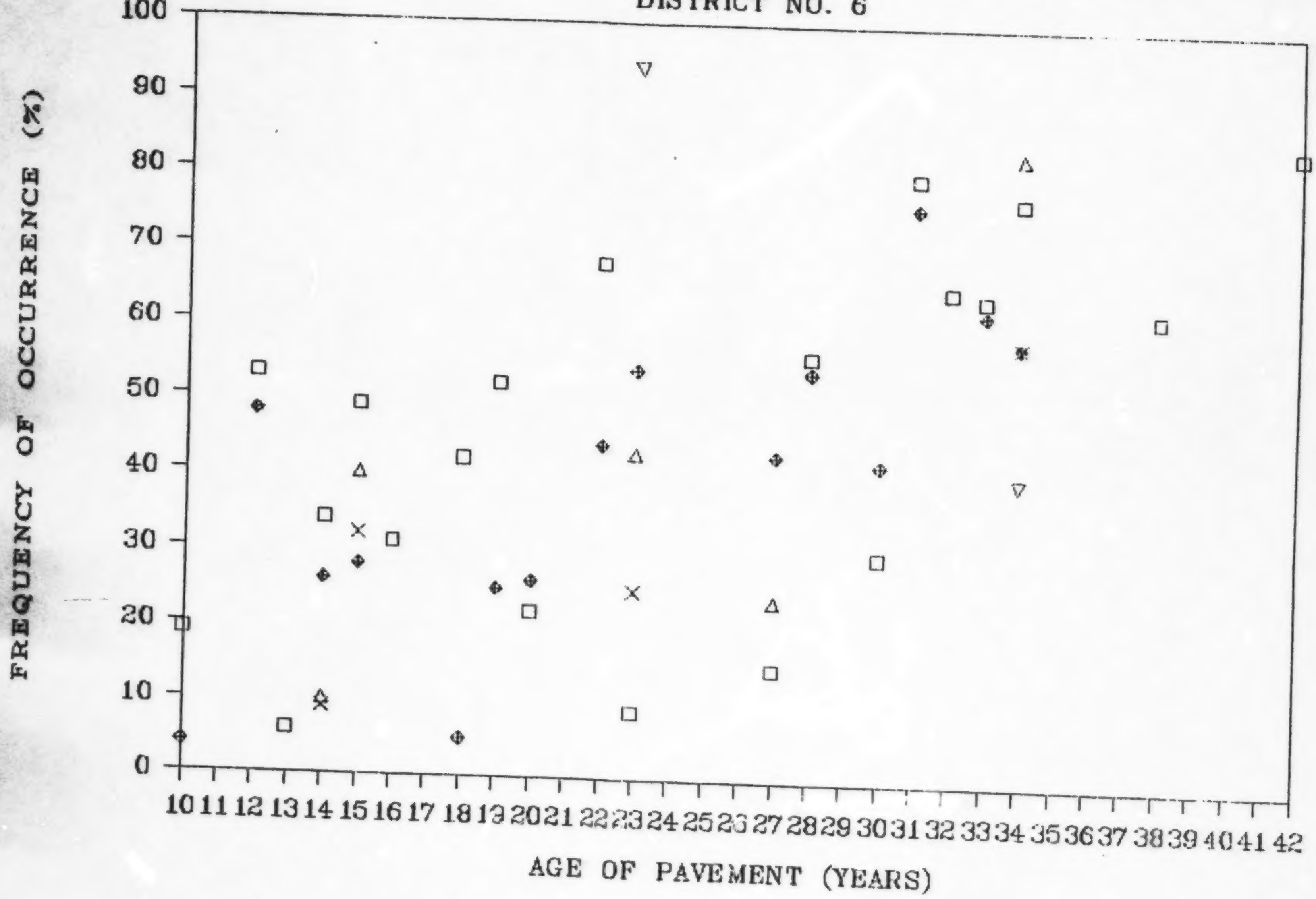
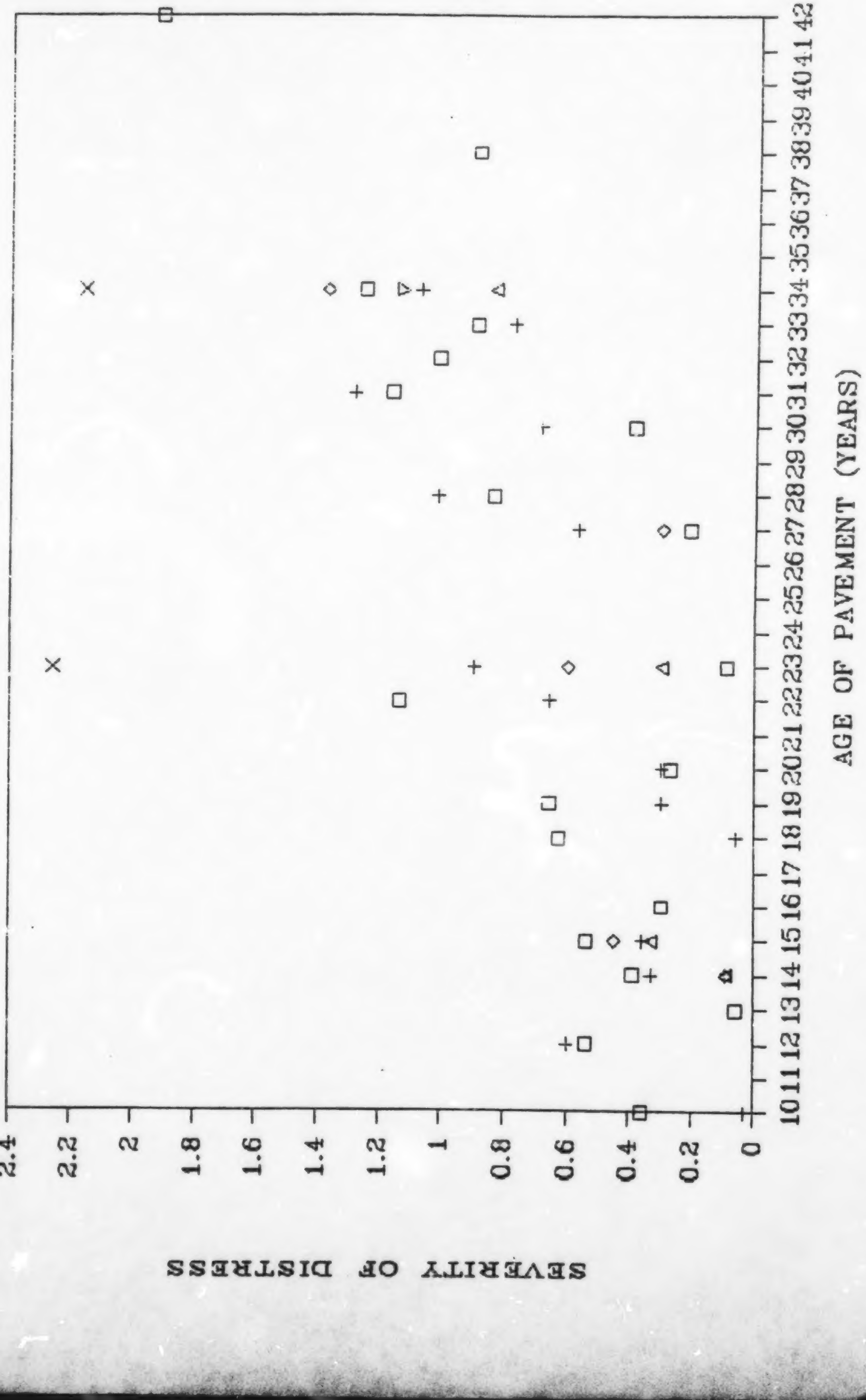


SEVERITY OF D--CRACKING

DISTRICT NO.6



FREQUENCY OF OCCURRENCE vs SEVERITY

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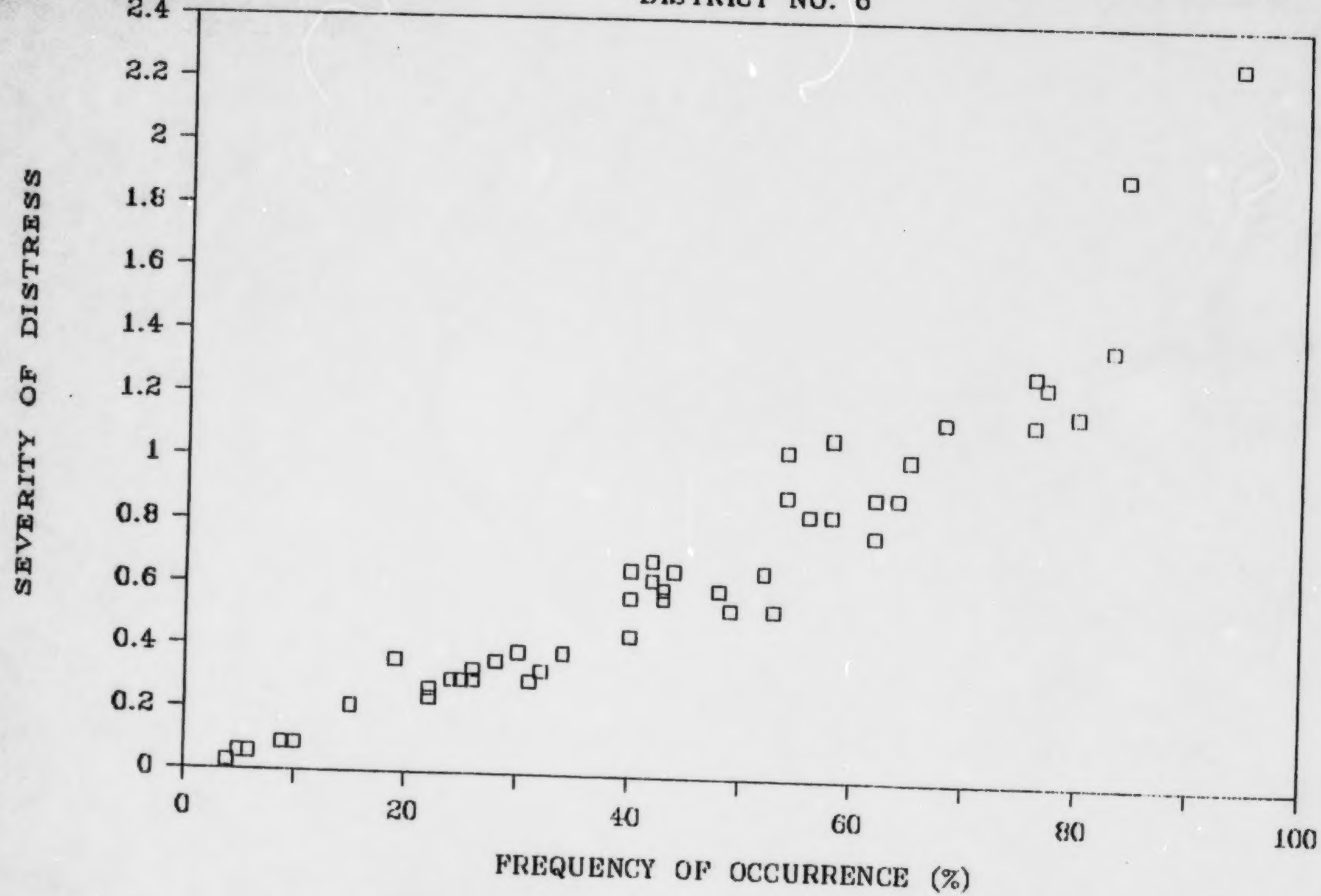
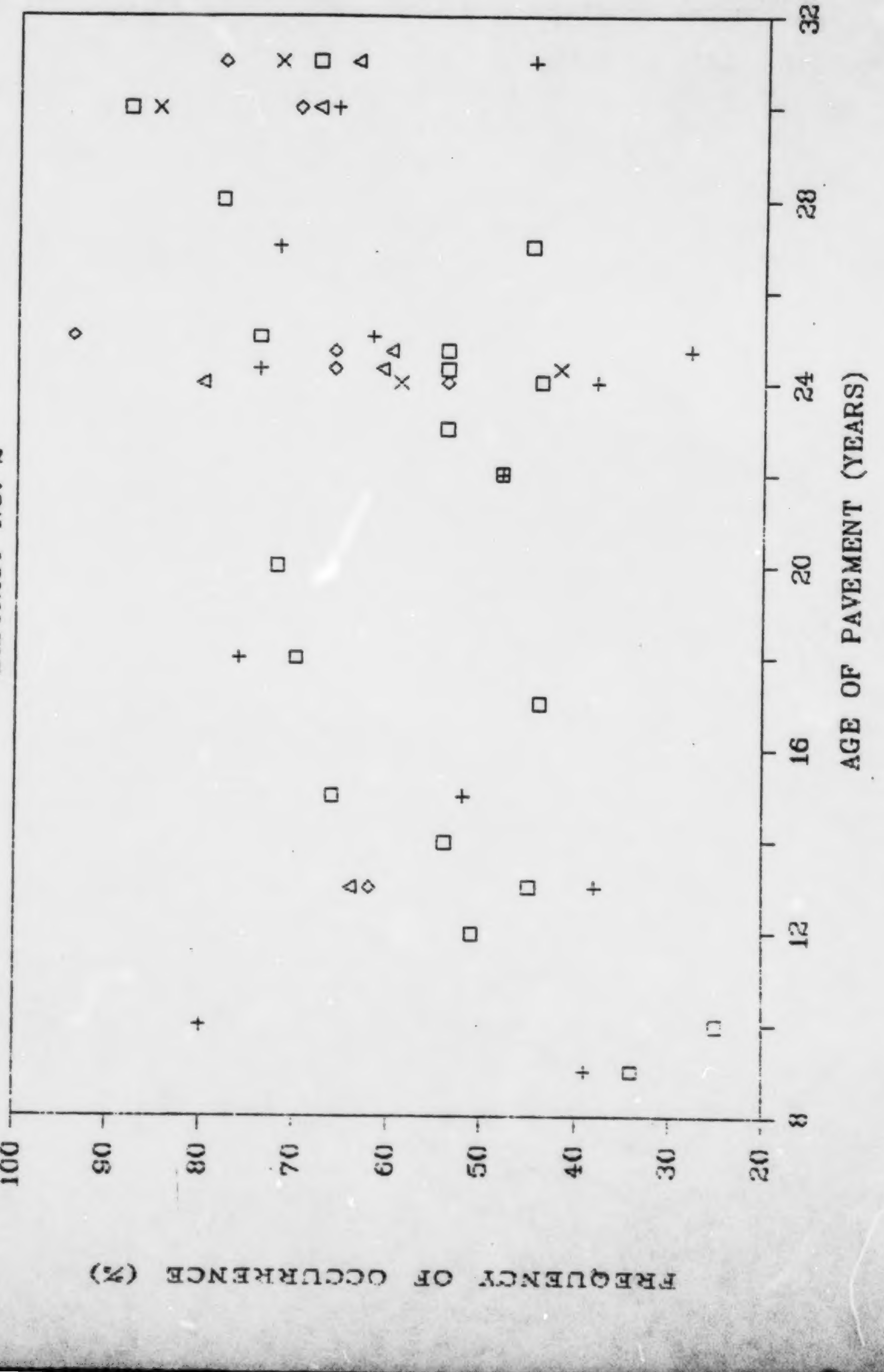


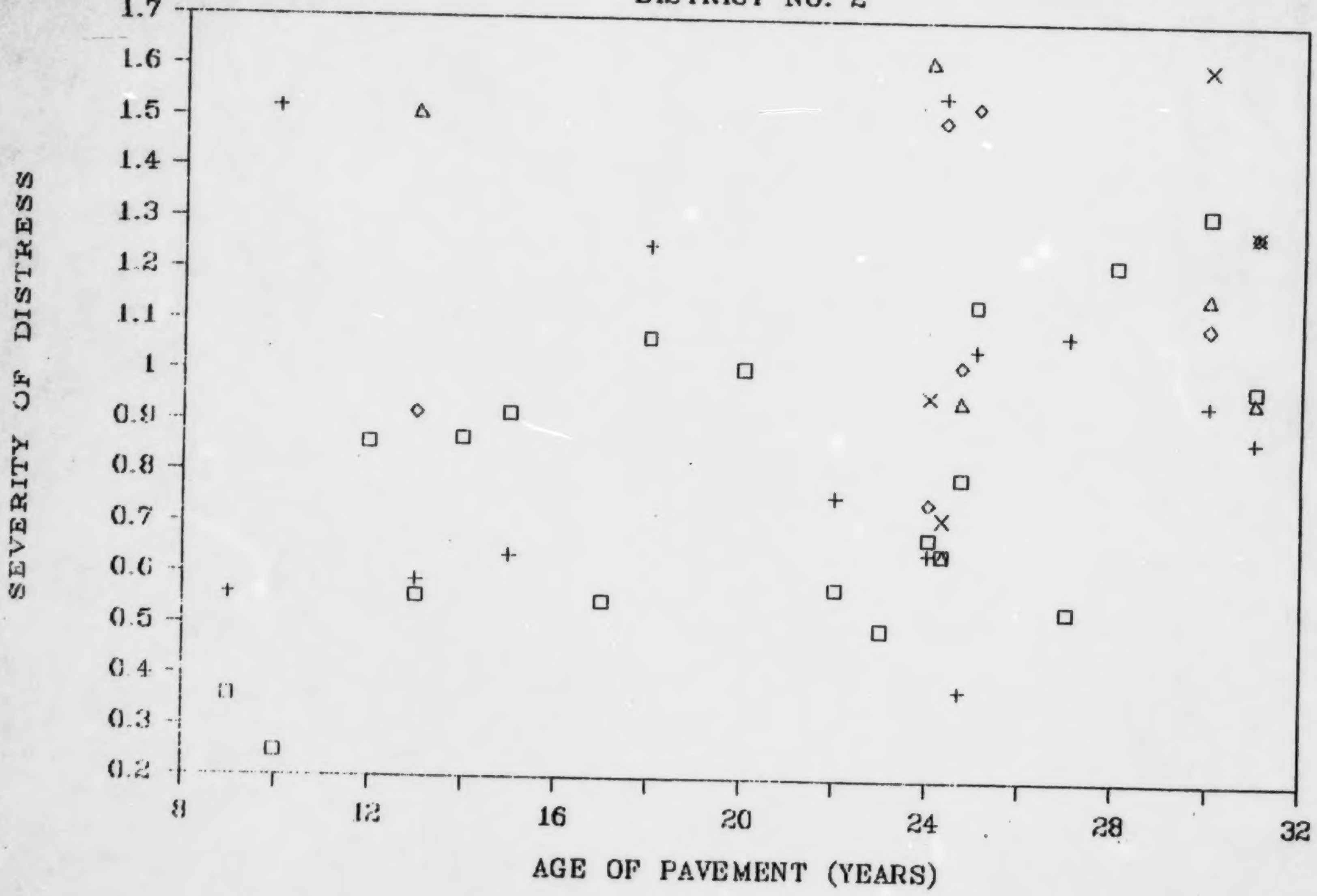


OCCURRENCE OF D—CRACKING

DISTRICT NO. 2

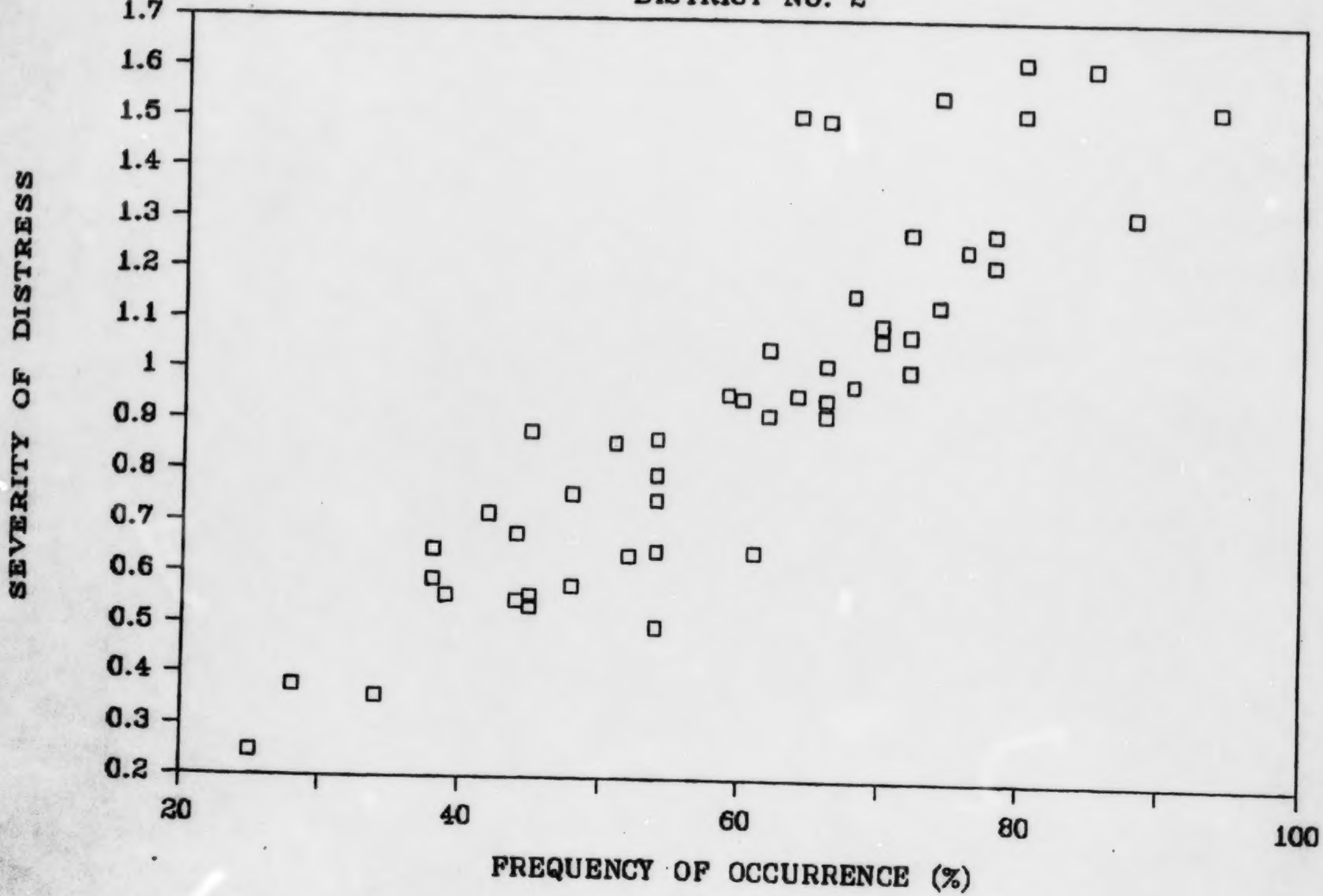
SEVERITY OF D-CRACKING

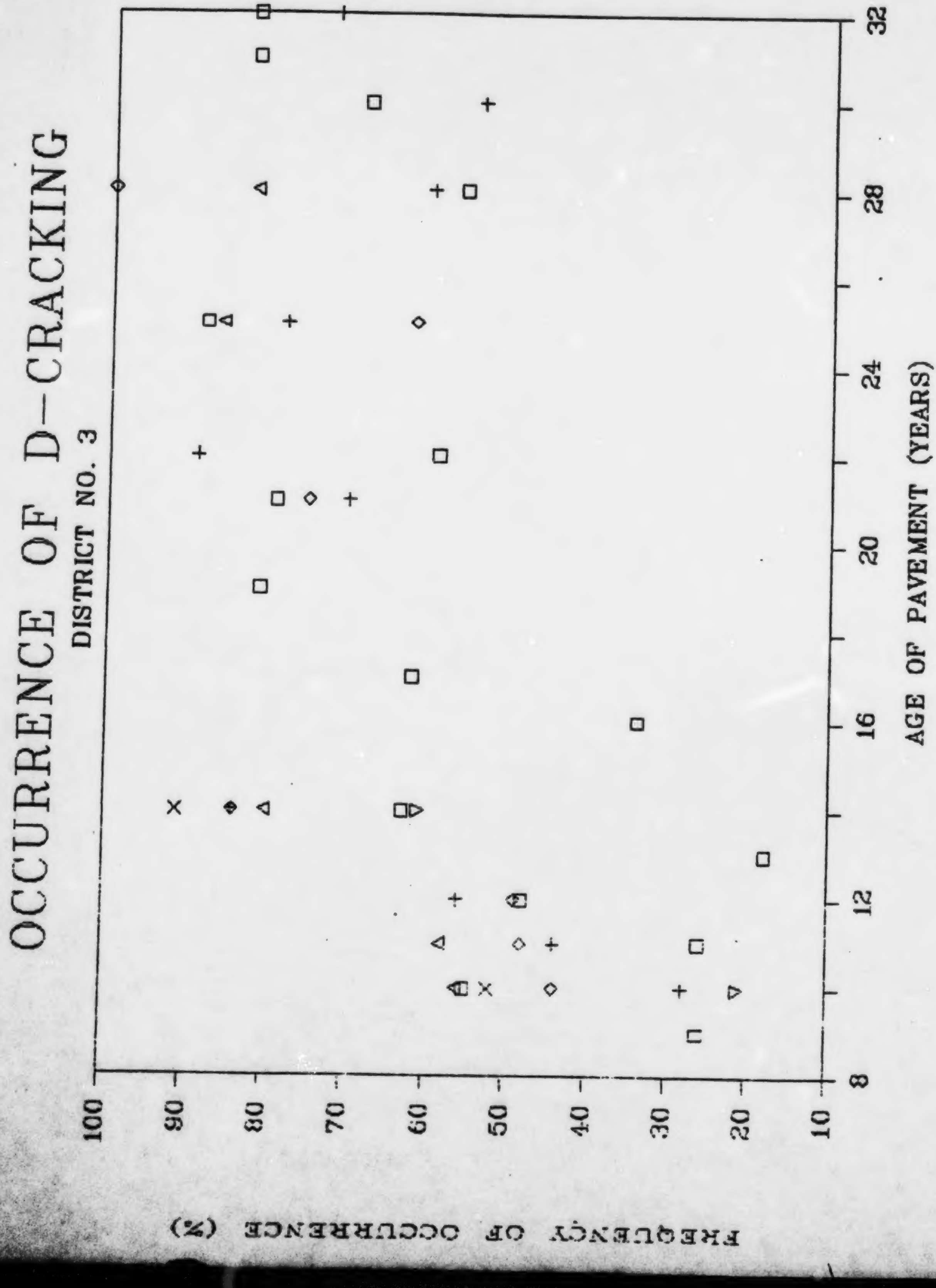
DISTRICT NO. 2



FREQUENCY OF OCCURRENCE vs SEVERITY

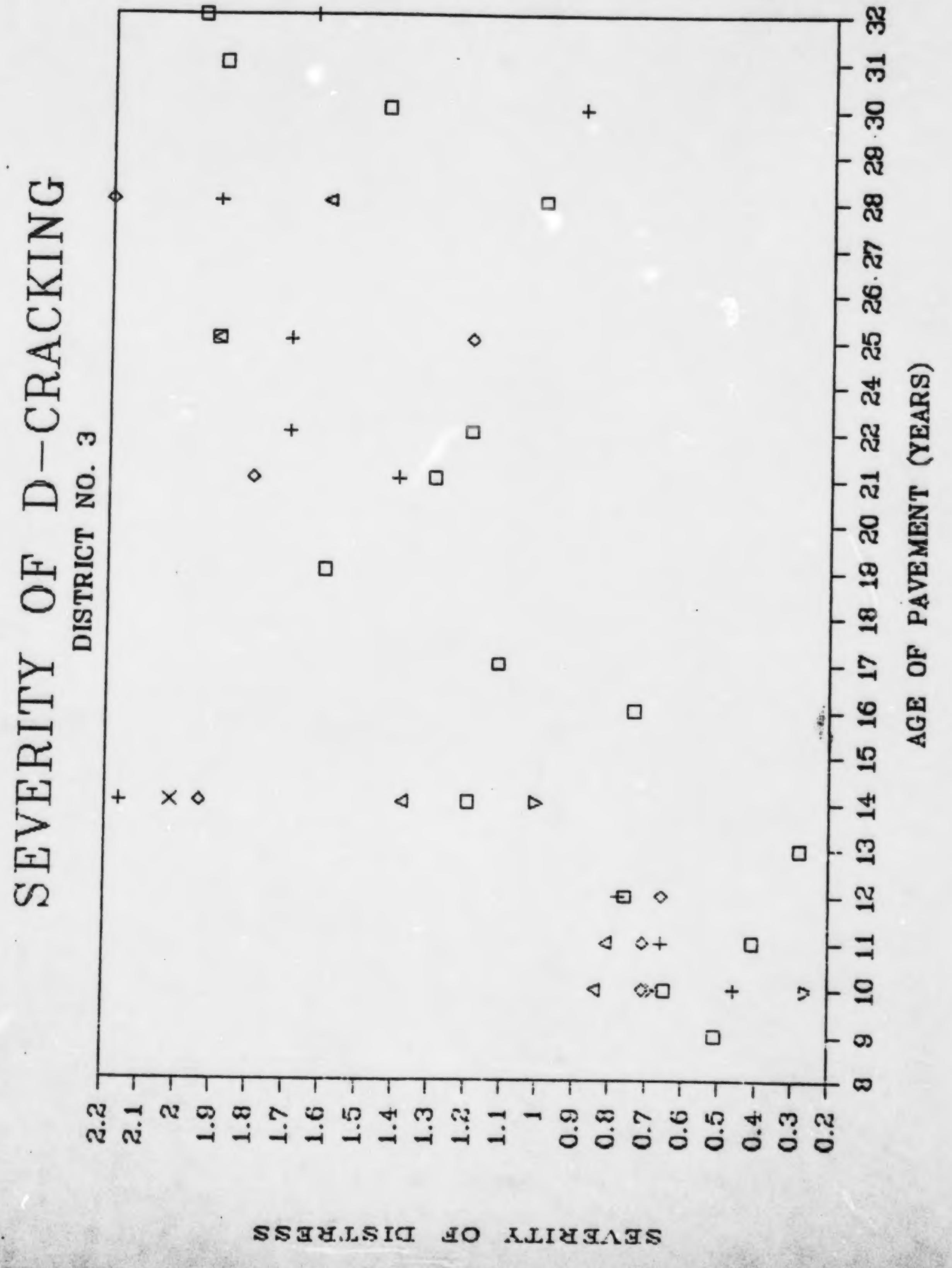
DISTRICT NO. 2





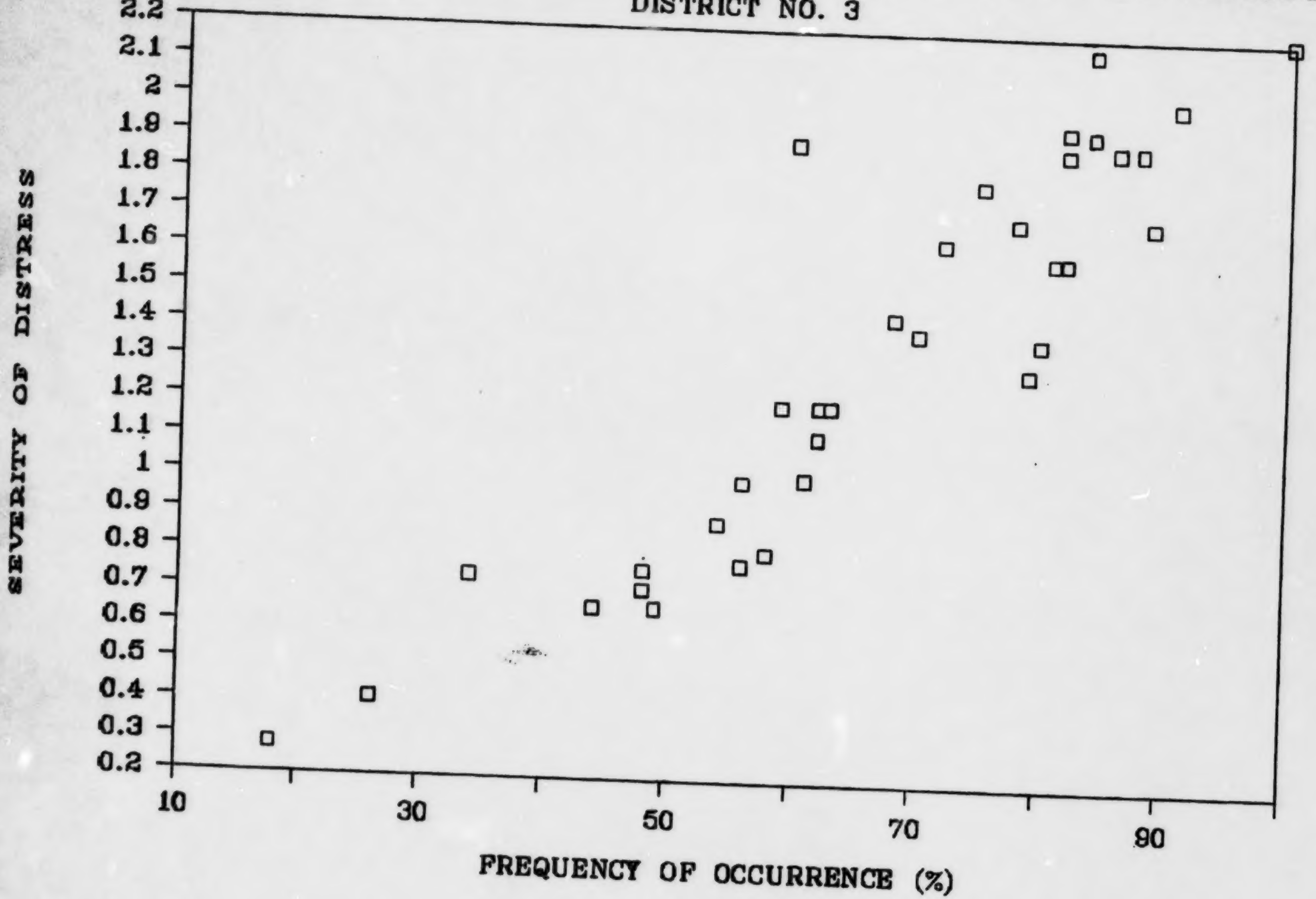
OCCURRENCE OF D—CRACKING

DISTRICT NO. 3


SEVERITY OF D-CRACKING
DISTRICT NO. 3
SEVERITY OF DISTRESS
2.2
2.1
2
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
AGE OF PAVEMENT (YEARS)
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
24
25
26
27
28
29
30
31
32

FREQUENCY OF OCCURRENCE vs -SEVERITY

DISTRICT NO. 3



FREQUENCY OF OCCURRENCE vs SEVERITY

DISTRICT NOS. 6,5,3,& 2